CHAPTER 6.0
EMBANKMENT SETTLEMENT

Embankment settlement is the most prevalent foundation problem in highway construction. Unlike stability problems, the results are seldom catastrophic but the cost of perpetual maintenance of continuing settlement are immense. The difficulty in preventing these problems is not as much a lack of technical expertise as a lack of communication between personnel involved in the roadway design and those involved in the structure design.

6.1 TYPICAL EMBANKMENT SETTLEMENT PROBLEMS

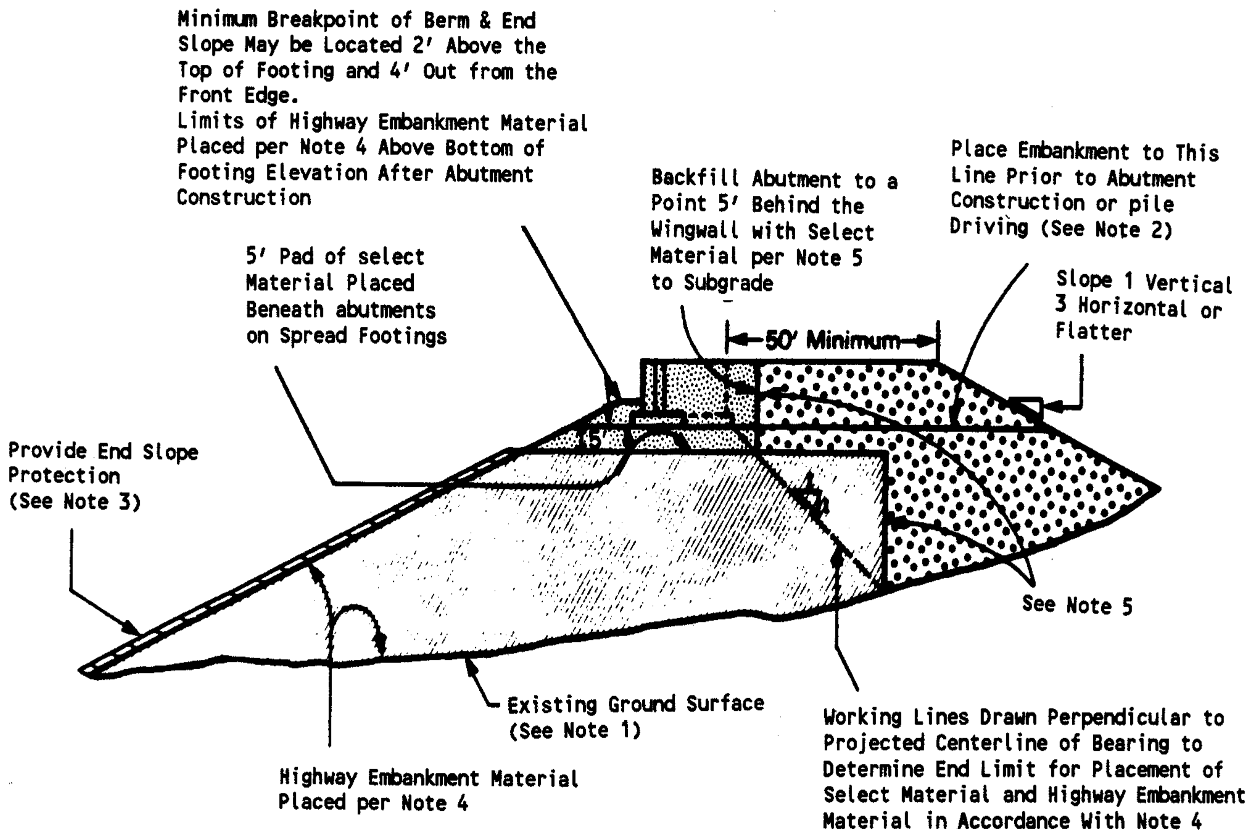
The design of a roadway embankment can utilize a wide range of soil materials and permit substantial amounts of settlement without affecting the performance of the highway. Roadway designers necessarily permit such materials to reduce project costs by utilizing cheap locally available soils. Structures are necessarily designed for little or no settlement to maintain specified highway clearances and to insure integrity of structural members. The approach embankment must affect a transition between roadway and structure while providing adequate structural foundation support. In most agencies the responsibility for approach embankment design is not defined as a structural issue, which results in roadway criteria being used across the structure. This is wrong; the approach embankment requires special materials and placement criteria to prevent internal consolidation and to moderate external consolidation.

6.2 COMMON DESIGN SOLUTIONS TO EMBANKMENT SETTLEMENT

6.2.1 Eliminate settlement within the approach embankment

A well constructed soil embankment, using quality control with regard to material and compaction, will not consolidate. Standard specifications and construction drawings should be prepared for the approach embankment area (normally designated to extend 50 feet behind the wingwall). The structural designer should have the responsibility for selecting the appropriate approach embankment cross section depending on selection of structure foundation type. A typical suggested approach embankment cross section is shown on Figure 6-1 for spread footing and pile foundations.

Special attention must be given to the interface area between the structure and the approach embankment, as this is where the famous "bump at the end of the bridge" occurs. The reasons for the bump are twofold; poor compaction of embankment material near the structure and migration of fine soil into drainage material. Poor densification is caused by restricted access of standard compaction equipment. Proper densification can be achieved by optimizing the soil gradation in this area to permit maximum density with minimum effort. Figure 6-2 shows a suggested detail for placement of drainage material. Typical specifications for select structure backfill and underdrain filter material to prevent the problem are included in Appendix E and F respectively. Similar results can be obtained by the use of prefabricated geocomposite drains which are attached to the backwall and connected to an underdrain.



Note 1: Topsoil shall be stripped beneath approach embankments less than 20' in height from a rectangular or trapezoidal area abounded by lines 15 feet outside the abutment and wingwall footings, or to the top of slope, whichever is less. The depth or stripping shall be determined by the soils Engineer and displayed on the highway cross sections by the Design Engineer.

Note 2: At some sites, fill is to be placed to the subgrade of the roadway and allowed to stand, in order to consolidate underlying material, before piles are driven.

Note 3: Slope protection treatment shall be as specified by the Bridge Engineer.

Note 4: Highway embankment material placed within these limits shall have a maximum dimension of 6 inches and shall be compacted to 95% of AASHTO T-180 maximum density. Quantity to be included in highway estimate.

Note 5: Highway embankment material and select material shall be placed concurrently on both sides of the vertical payment line.

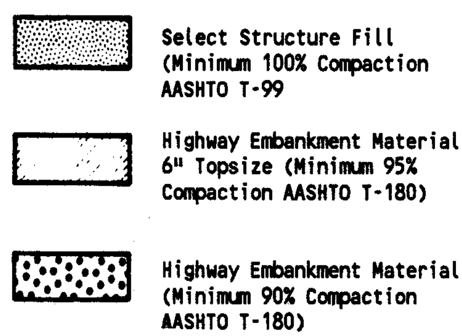


Figure 6-1: Suggested Approach Embankment Details

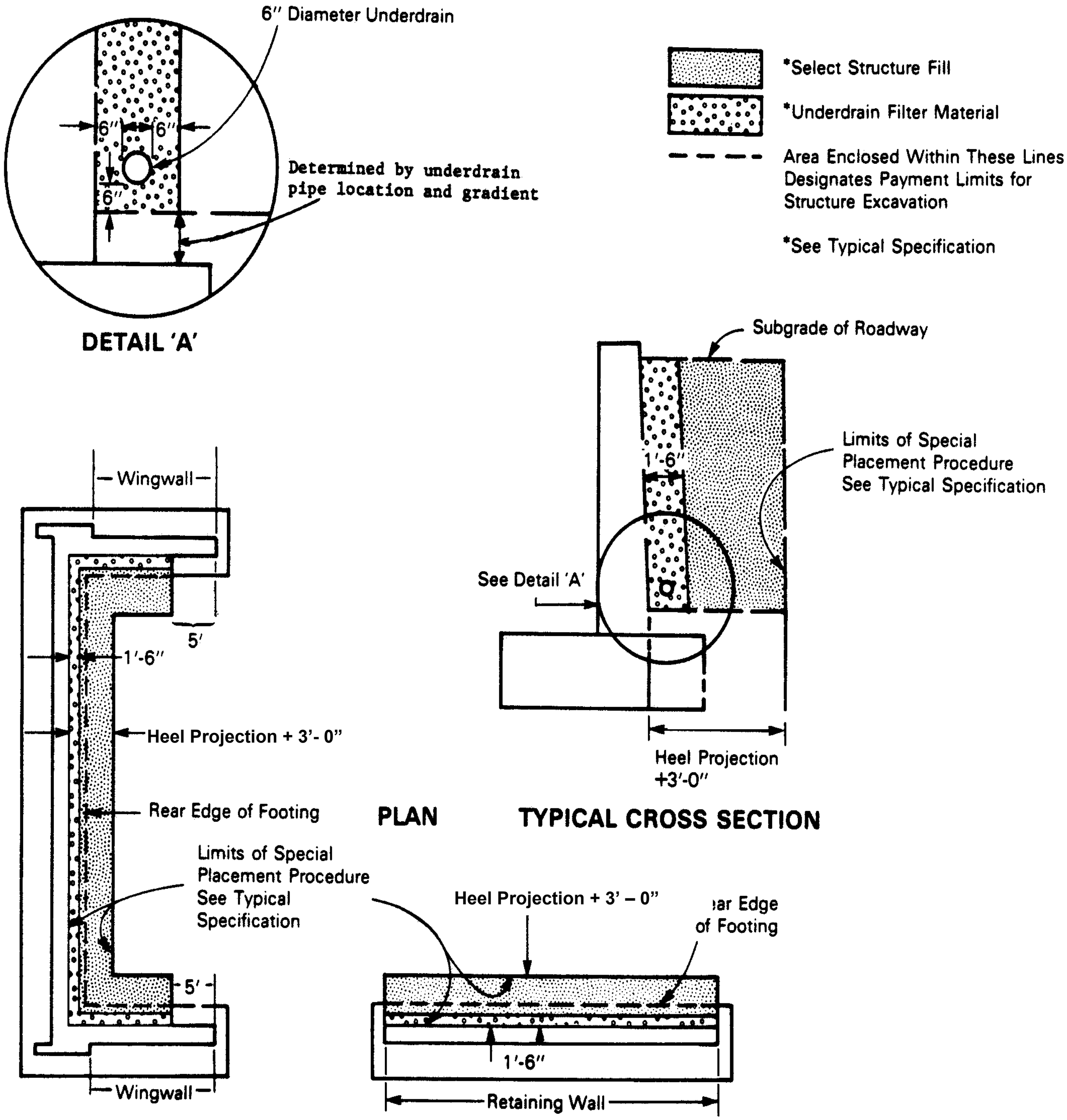


Figure 6-2: Structure backfill placement limits for porous drainage aggregate.

6.2.2 General Consideration for Select Structure Backfill

Select structure backfill is usually placed in relatively small quantities and in relatively confined areas. Structure backfill specifications must be designed to insure construction of a durable, dense backfill. The following considerations (Table 6-1) must be addressed:

Table 6-1
General Considerations for Select Structural Backfill

Consideration	Reason For
Lift Thickness	6" to 8", so compaction possible with small equipment
Topsize	Less than ¾ of lift thickness
Gradation	Well graded for ease of compaction
Durability	Minimize breakdown of particles and settlement
Percent Fines	Minimize to prevent piping and allow rapid drainage
T99 Density Control	Small equipment cannot achieve AASHTO T180 densities
Compatibility	Particles should not move into voids of adjacent fill or drain material

6.2.3 Estimate Settlement of the Approach Embankment Caused by Consolidation of the Subsoil

Many and varied procedures exist for computation of embankment settlement. Two methods will be presented herein; one each for cohesionless and cohesive soils. However, certain steps are common to either method, namely pressure distribution.

6.3 GENERAL PROCEDURE FOR APPROACH EMBANKMENT PRESSURE DISTRIBUTION

1. Plot soil profile including soil unit weights, SPT results (N), moisture contents and interpreted consolidation test values.

2. Draw overburden pressure (P_o) diagram with depth.

3. Plot total embankment pressure (P_F) on the P_o diagram at ground surface level.

4. Distribute the total embankment pressure with depth using appropriate pressure coefficient charts. Figure 6-3 is a chart used for distribution of pressure beneath an approach embankment and end slope.

The fundamental principles to remember are that stresses from an embankment load spread out with depth in proportion to the embankment width and that the additional pressures on the soil decrease with depth.

6.3.1 Pressure Distribution Chart Use

Step 1. Determine the distance (b) from the centerline of the approach embankment to the midpoint of sideslope. Multiply the numerical value of "b" by the appropriate values shown on the right vertical axis of the chart to develop the depth at which the distributed pressures will be computed.

Step 2. Select the point (X) on the approach embankment where the settlement prediction is desired (normally at the intersection of the centerline of the embankment and the abutment). Measure the distance from this point X to the midpoint of the end slope. Return to the chart and scale that distance on the horizontal axis from the appropriate side of the midpoint of end slope line.

Step 3. Read vertically down from the plotted distance to the various curves corresponding to depth below surface. The "k" value on the left vertical axis should be read and recorded on a computation sheet with the corresponding depth.

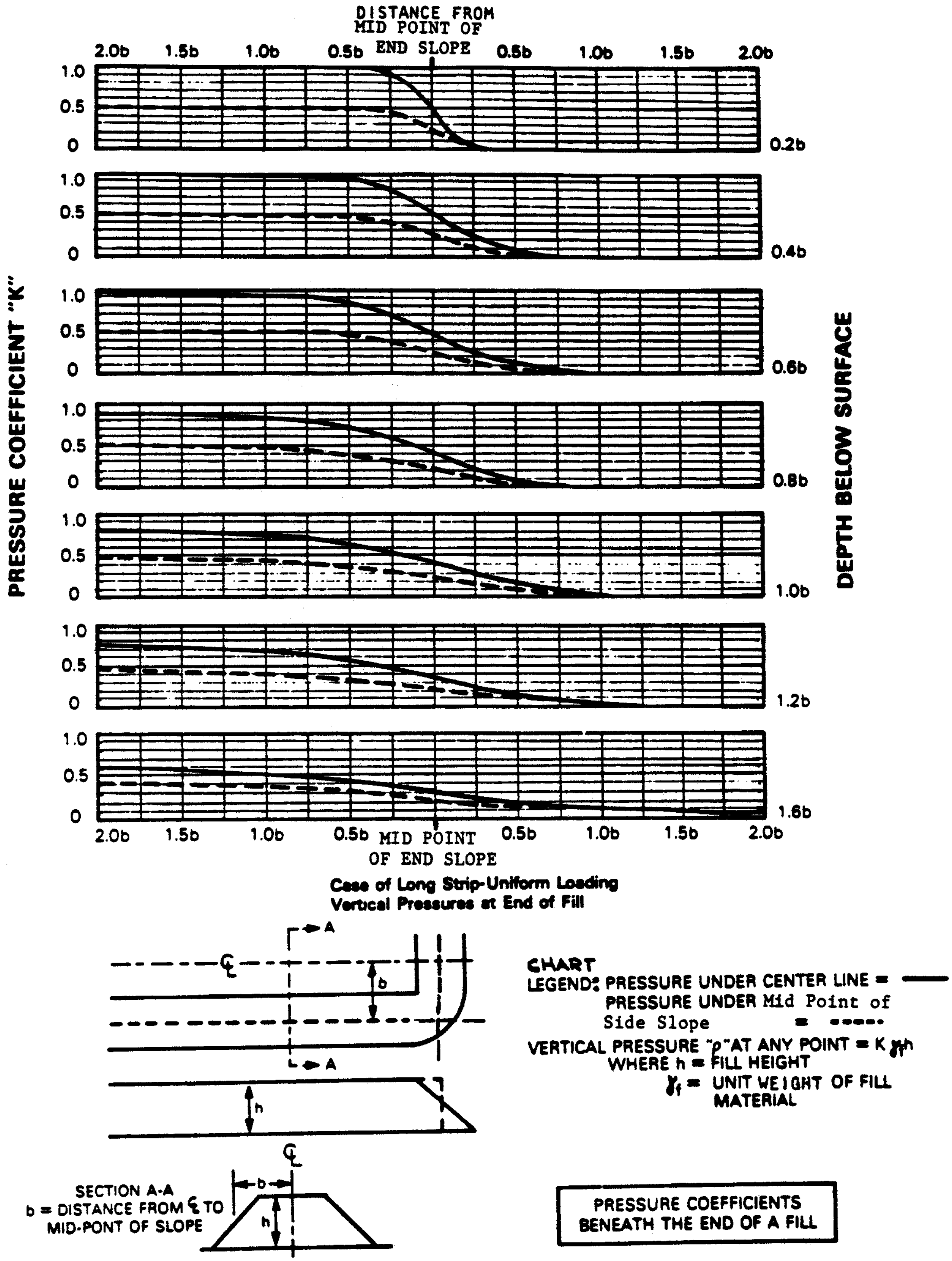


Figure 6-3: Pressure coefficients beneath the end of a fill

Step 4. Multiply each "k" value by the value of total embankment pressure to determine the amount of pressure (ΔP) transmitted to each depth. The ΔP values should be added to the P_o values at each depth to determine the final pressure (P_F). The P_F values should then be plotted on the P_o diagram and connected to form the P_F line as follows:

Use Figure 6-3 charts (0.2B, 0.4B, etc) to find k values at depths from 0′ - 100′ ±. Multiply the k values times the embankment pressure P_0 to find ΔP at depths of 0.2B, 0.4B, etc. Add ΔP to P_o at those depths and connect the points to produce a plot of P_F as shown in Figure 6-4.

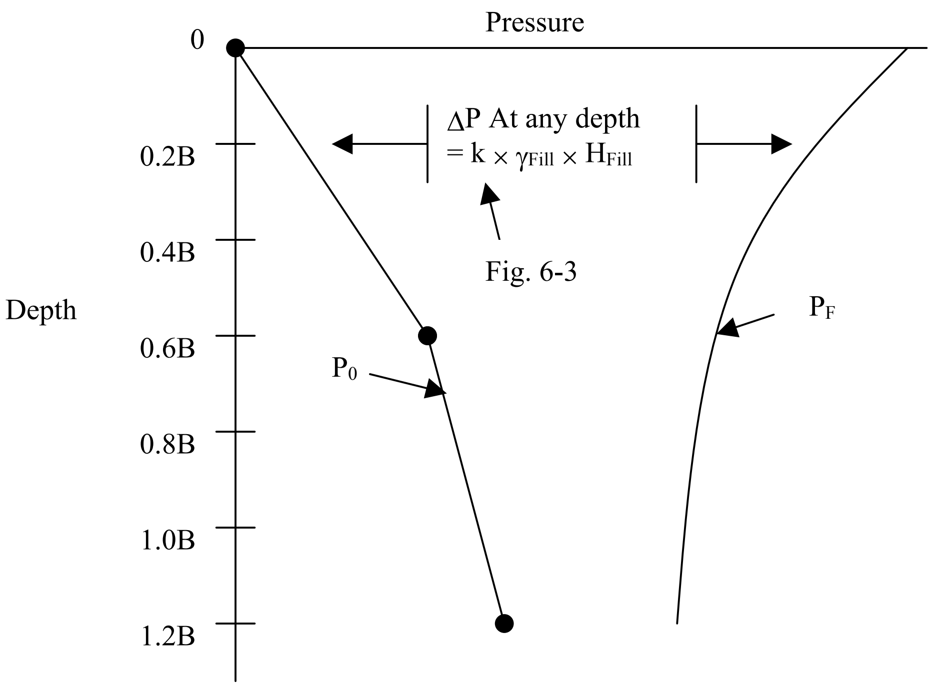
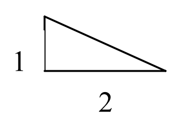


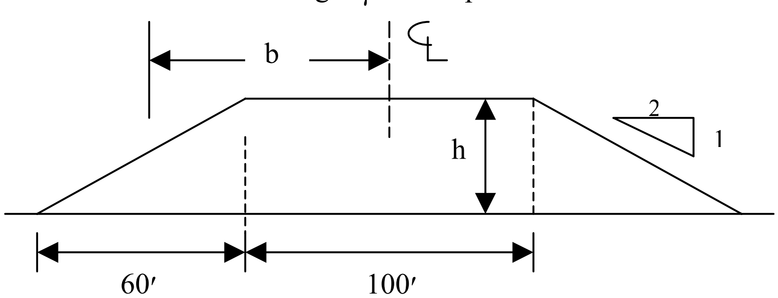
Figure 6-4: Plot of Pressure Increase with Depth Below an Embankment

Example 6-1 - Use of Pressure Distribution Chart (Fig. 6-3)

Given: Fill height h = 30 ft.
End and side slopes (1V:2H)



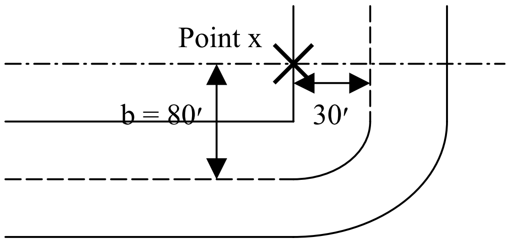
Embankment top width = 100 ft.
Fill unit weight γ_F = 100 pcf



Distance from centerline (℄) to mid point of side slope $b = \frac{100}{2} + \frac{60}{2} = 80'$

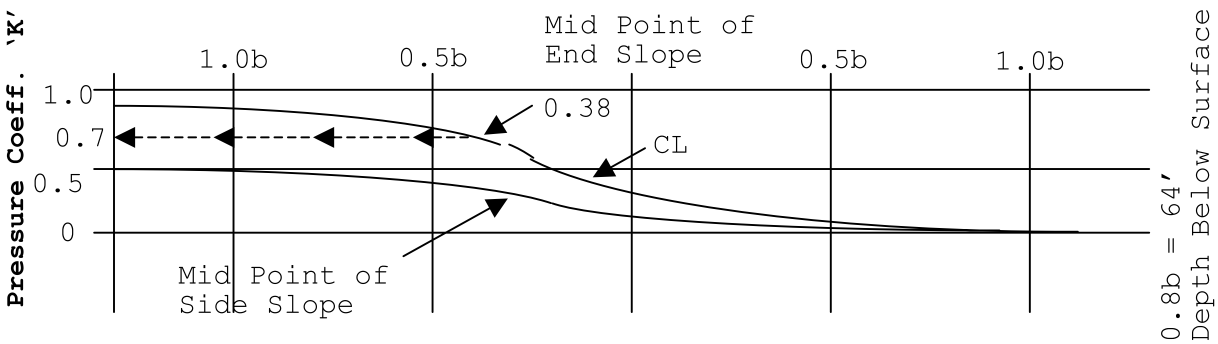
Find: The pressure increase (ΔP) under the proposed abutment centroid (point x) at a depth of 0.8b (64ft).

below the base of the fill.



Solution: Distance from midpoint of end slope to point ‘x’ = 30′. ENTER PRESSURE DISTRIBUTION CHART FOR 0.8b depth at $\frac{30b}{80} = 0.38b$ distance from MIDPOINT OF END SLOPE.

Pressure Distribution Chart



From 0.8b (64′) depth chart read k = 0.7

$\therefore$ at 64′ depth $\Delta P = k\ \gamma_F\ h = (0.7)(100\ \text{pcf})(30\ \text{ft.})$

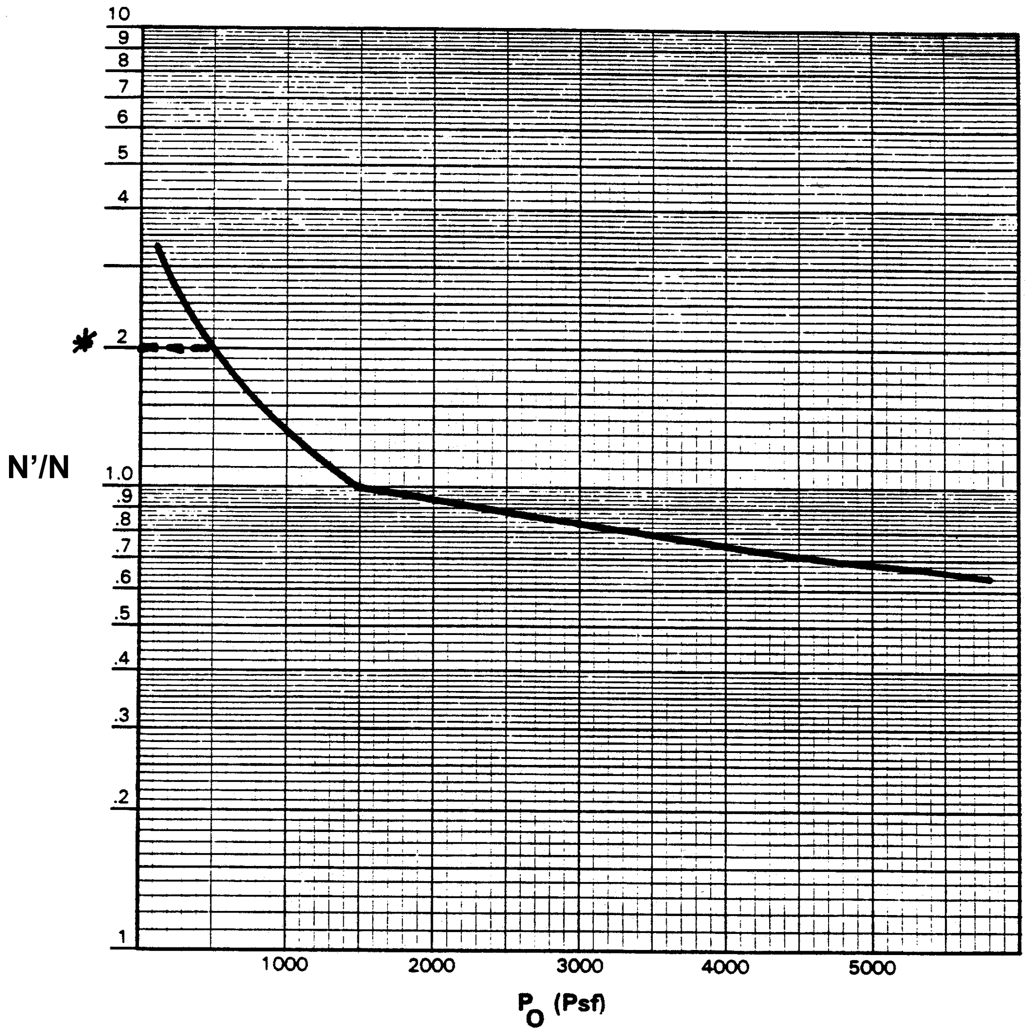
$\Delta P = 2100$ psf

($\Delta P'_s$ at other depths found from other ″b″ charts)

6.4 SETTLEMENT COMPUTATION FOR COHESIONLESS SOILS

6.4.1 Correction of SPT Blow Counts

In recent years much attention has focused on the validity of field SPT blow counts (N). Numerous factors which influence SPT counts with increasing depth were investigated by field testing. The conclusion of that testing showed that physical factors such as increasing drill rod weight or rod flexibility had a minor overall effect on N counts. However, in non-cohesive soils, the increasing overburden pressure resulted in N values at increasing depths which indicated larger relative densities than actually existed. Conversely, at very shallow depths, where overburden pressures are low, N values indicated lower relative densities than actually existed. These overburden effects must be considered if correlations are to be made between N values and physical soil properties such as unit weight and friction angle. Therefore, N values from field operations must be corrected by the designer to reflect overburden pressure changes. Figure 6-5 should be used to obtain corrected SPT values, N′. In practice the maximum correction value should not exceed 2.



Where: N' = Corrected SPT Value Blow Count
N = SPT Value
Po = Existing Effective Vertical Overburden Pressure
* = Suggested Maximum Value

Reference: Based on 1967, Bazaraa, The Use of Standard Penetration Test for Estimating Settlement of Shallow Foundation on Sand

Figure 6-5: Correcting SPT (N) blow counts for overburden pressure, P_o

Step 1. Determine corrected SPT value (N′) from Figure 6-5.

Step 2. Determine Bearing Capacity Index (C′) by entering Figure 6-6 with N′ value and the visual description of the soil,

Step 3. Compute settlement in 10′ ± increments of depth from

$$\Delta H = H\left(\frac{1}{C'}\right)\text{Log}\frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}1)$$

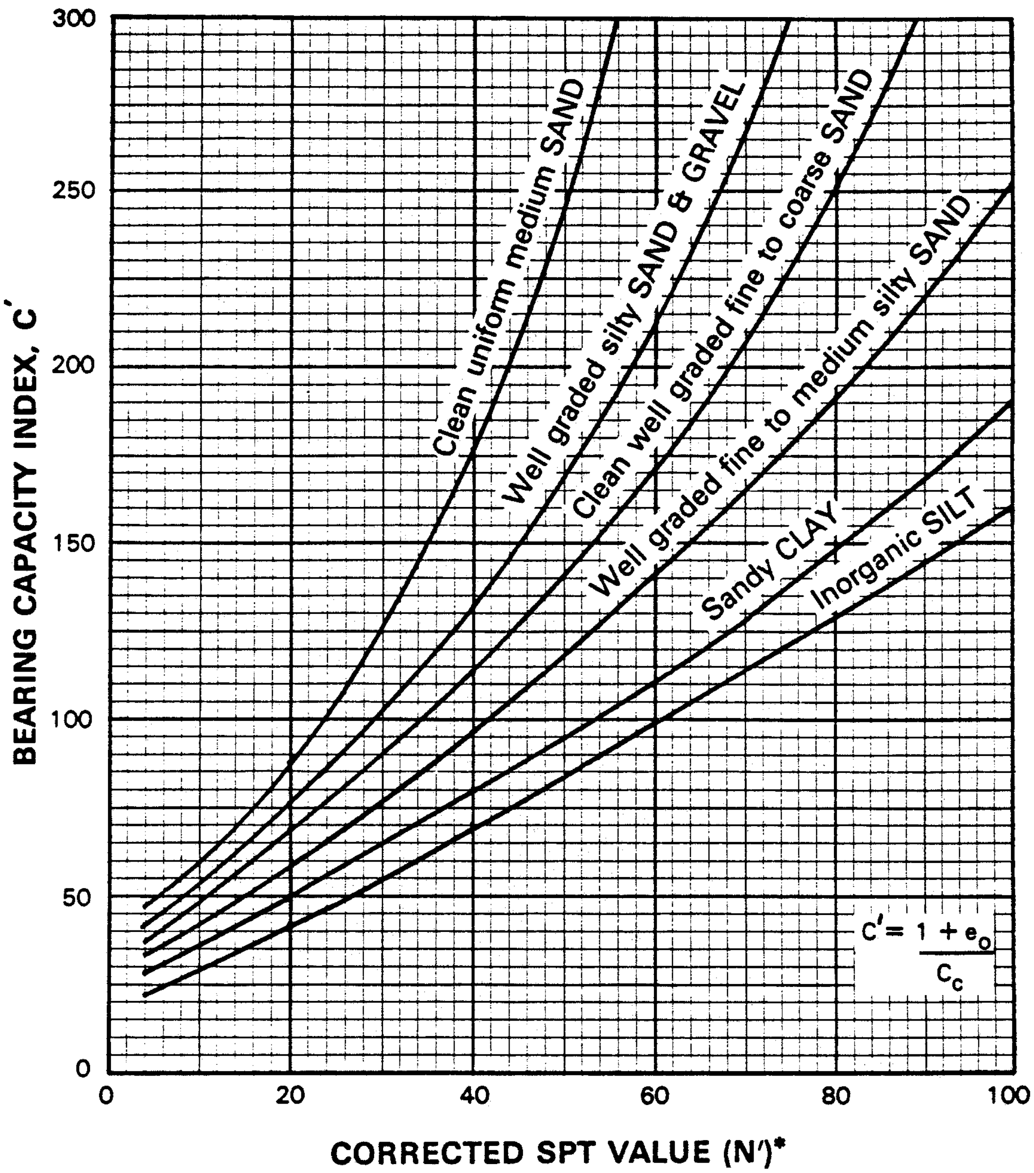
Where:
- ΔH = Settlement (Feet)
- H = Thickness of soil layer considered (Feet)
- C′ = Bearing capacity index (Figure 6-6)
- P_o = Existing effective overburden pressure (psf) at center of considered layer. For

shallow surface deposits, a minimum value of 200 psf must be used to prevent unrealistic computation of settlement.

ΔP = Distributed embankment pressure (psf) at center of considered layer

P_F = Final pressure felt by foundation subsoil (psf)

Note: $P_F = P_o + \Delta P$



*N'—*SPT (N) Value Corrected for Overburden Pressure.*

Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

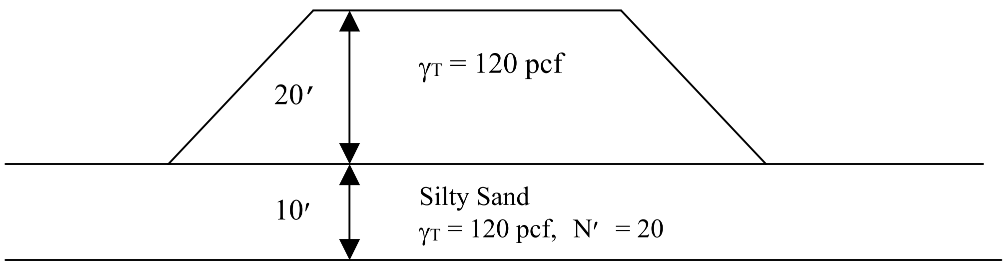
Figure 6-6: Bearing capacity index (C') values for granular soils

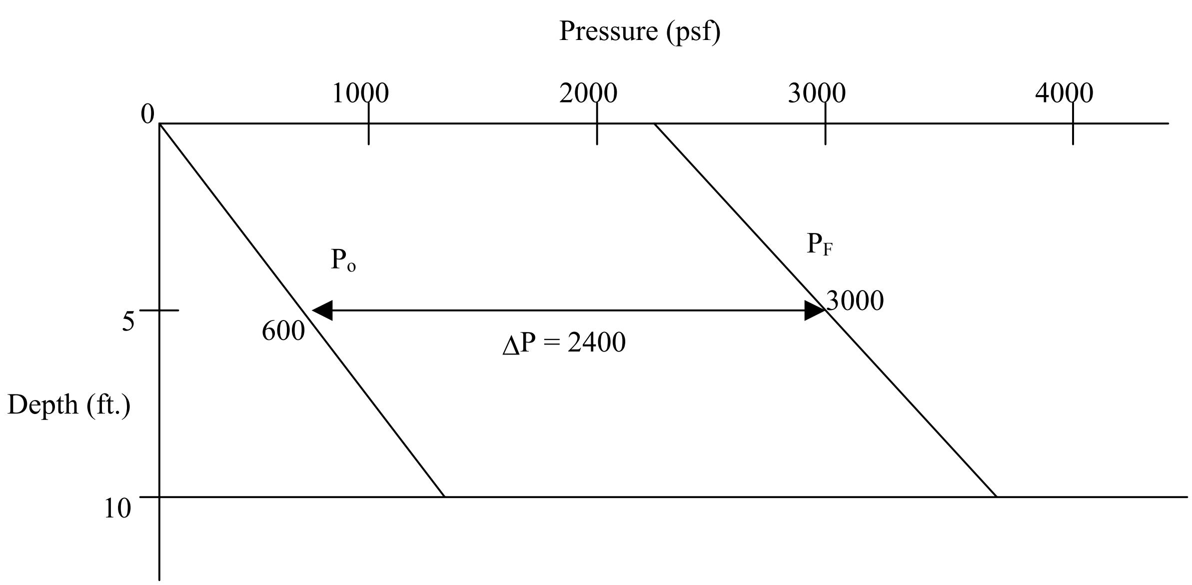
6.4.2 Time for Settlement

Time rate of settlement is not a concern for cohesionless soils. Cohesionless soils, being highly permeable, will settle instantaneously as load is applied. Embankment settlement amounts caused by consolidation of cohesionless soil deposits are frequently ignored because the settlement amounts are small in relation to cohesive deposits and the settlements occur as the embankment is constructed.

Time rate of consolidation settlement for cohesive soils is discussed in Section 6.7

Example 6-2: Determine The Settlement Of The Embankment Due To Consolidation Of The Silty Sand Layer Using The P_O Diagram.





Solution

Find C′: Use N′ = 20 and Silty Sand Curve
In Figure 6-6
C′ = 58

Find Settlement

$$\Delta H = H\frac{1}{C'}\,\text{Log}\frac{P_0 + \Delta P}{P_0} \qquad (6\text{-}1)$$

$$\Delta H = 10'\left(\frac{1}{58}\right)\text{Log}\ \frac{600\ \text{psf}\ + 2400\ \text{psf}}{600\ \text{psf}}$$

$$\Delta H = 0.12' = 1.44''$$

6.5 SETTLEMENT COMPUTATION FOR COHESIVE SOILS

1. Analyze consolidation test data to determine:

   a. Preconsolidation pressure (P_c)

   b. Initial void ratio (e_o) at P_o

   c. Compression and recompression indices (C_c and C_r)

1. (ALT.) In the absence of consolidation test data, settlement may be approximated using Atterberg limit and moisture content data. This method is only recommended for use in final design if soils exist which are not suited for lab testing, i.e., surface muck deposits, etc.

   a. Soil may be assumed to be preconsolidated to pressures above typical embankment loadings if the liquidity index ([moisture content minus plastic limit] divided by plastic index) is less than 0.7.

   b. Initial void ratio, e_o, for saturated soils may be determined by multiplying the moisture content by the specific gravity and dividing by 100.

   c. C_c and C_r may be determined by dividing the moisture content by 100 and 1,000 respectively.

Example 6-3: Given moisture content 30, liquid limit 50, plastic limit 25, specific gravity 2.75. Find e_0, C_c, and C_r and determine if the soil is preconsolidated.

Solution:

Liquidity index = $\frac{30-25}{50-25} = 0.2$ (preconsolidated, see "a" above)

$$e_o = \frac{(2.75)(30)}{100} = 0.825$$

$$C_r = \frac{30}{1000} = 0.03$$

$$C_c = \frac{30}{100} = 0.30$$

2. Compute settlement in 10′ ± increments of depth or at soil layer boundaries using:

$$\Delta H = H\left(\frac{C_c}{1+e_0}\right)Log\frac{P_0+\Delta P}{P_0} \tag{6-2}$$

(For normally consolidated soils only, see later sections for preconsolidated soils)

Where:
- ΔH = Settlement (feet)
- H = Thickness of soil layer considered (feet)
- C_c = Compression index (from consolidation test)
- C_r = Recompression index (from consolidation test)
- e_o = Initial void ratio of soil
- P_o = Existing effective overburden pressure (psf) at center of considered layer. For shallow surface deposits, a minimum value of 200 psf must be used to prevent unrealistic computation of settlement
- ΔP = Distributed embankment pressure (psf) at center of considered layer
- P_F = Final pressure (psf) felt by foundation subsoil
- P_F = $P_o + \Delta P$

Both the compression index, C_c, and the recompression index, C_r, may be used in settlement computations for preconsolidated clays. Only C_c is used in settlement computations for normally consolidated clays.

6.5.1 Normally Consolidated Clay

For normally consolidated clays, the preconsolidation pressure P_c is approximately equal to the existing overburden pressure P_o. This means that the soil has never in the past been loaded to a stress above that which presently exists in the ground, i.e., $P_o = P_c$.

$$\therefore \Delta H = H\frac{C_c}{1+e_0}Log\frac{P_F}{P_0} \tag{6-2a}$$

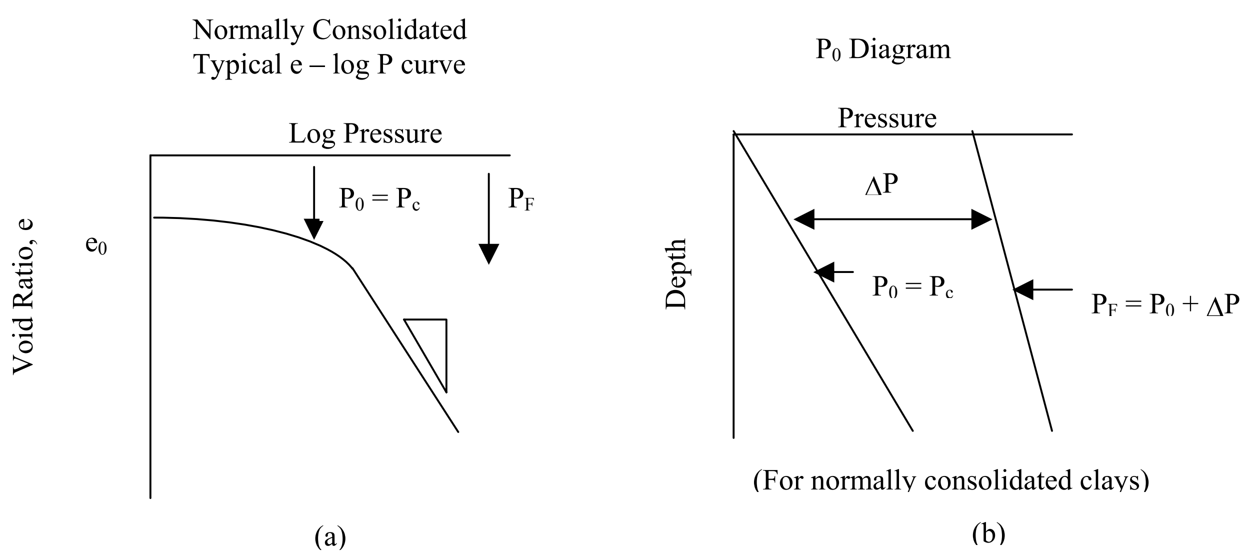


Figure 6-7: (a). Typical e-log P curve for Normally Consolidated Clay and, (b). Overburden Pressure (P_0) and Final Pressure Variation with Depth.

6.5.2 Preconsolidated Clay

The computation procedure for estimating settlement when preconsolidated clays exist in the soil profile is slightly more complicated. The computation is made much easier by use of the P_o diagram (Figure 6-8). For preconsolidated clays, the preconsolidation pressure P_c (determined from consolidation test) will be greater than the existing overburden pressure P_o (Figure 6-8). This means that at some time in the past the clay has been subjected to a greater stress than now exists (due to weight of glaciers, weight of soil that has since eroded away, or due to desiccation).

For $P_F < P_c$

$$\Delta H = H\frac{C_r}{1+e_0}\operatorname{Log}\frac{P_F}{P_0} \tag{6-2b}$$

For $P_F > P_c$

$$\Delta H = H\frac{C_r}{1+e_0}\operatorname{Log}\frac{P_c}{P_0} + H\frac{C_c}{1+e_0}\operatorname{Log}\frac{P_F}{P_c} \tag{6-2c}$$

These settlement analyses may be varied to judge the effects of excavation of unsuitable material, the placing of surcharges, or the substitution of lightweight fill materials.

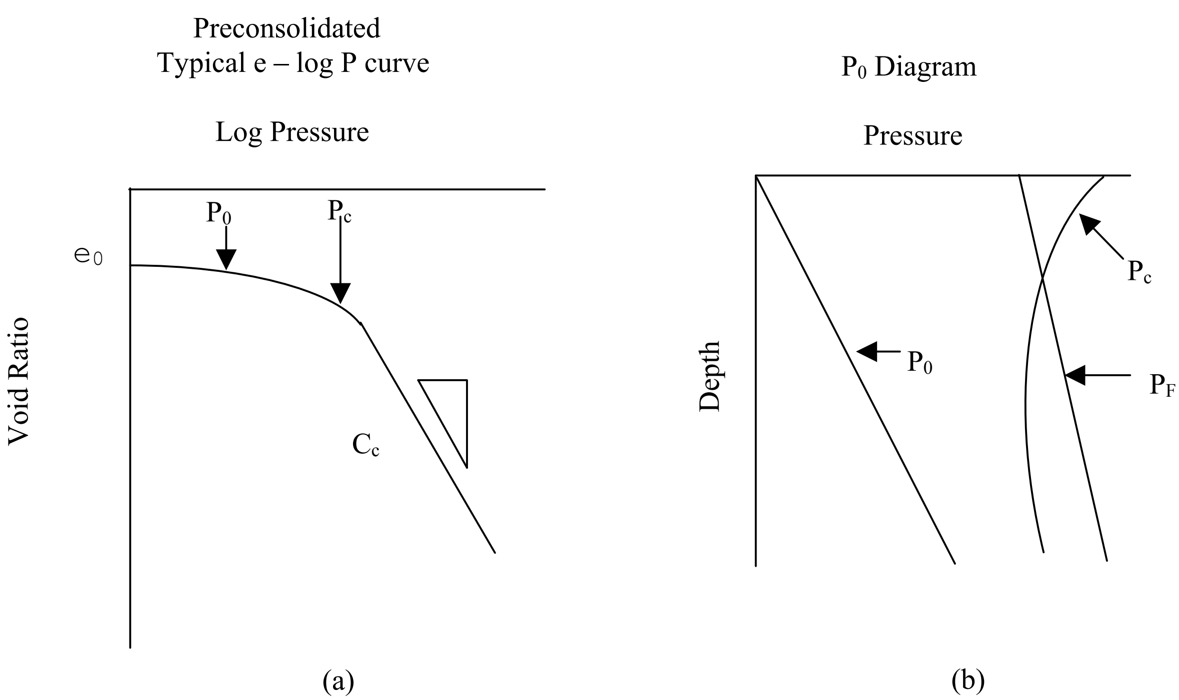


Figure 6-8: (a). Typical e-log P curve for Preconsolidation Clay and, (b). Variation of Overburden Pressure (P_0), Preconsolidation Pressure (P_c) and Final Pressure (P_F) with Depth.

6.6 ESTIMATING SECONDARY SETTLEMENT

Secondary settlement is of practical importance in soils containing organic material. Secondary settlement can occur for many years following construction. The "roller coaster" roadway that is typical for roads built across peat swamp deposits is sometimes due to long-term secondary settlements.

Secondary settlement can be estimated using the following relationship:

$$\Delta H_{sec} = C_{\alpha}\ H\ \text{Log}\frac{t_{sec}}{t_p} \tag{6-3}$$

Where:
- ΔH_{sec} = Secondary settlement
- C_{α} = Coefficient of secondary consolidation (determined from lab consolidation test)
- H = Soil layer thickness
- t_{sec} = Time over which secondary settlement is being estimated
- t_p = Time for primary consolidation

Typically, the ratio t_{sec}/t_p is taken as 10 when making the secondary settlement computation.

Approximate correlation of C_{α} versus natural water content is shown in Figure 6-9. The chart can be used to make secondary settlement estimate in the absence of consolidation test data.

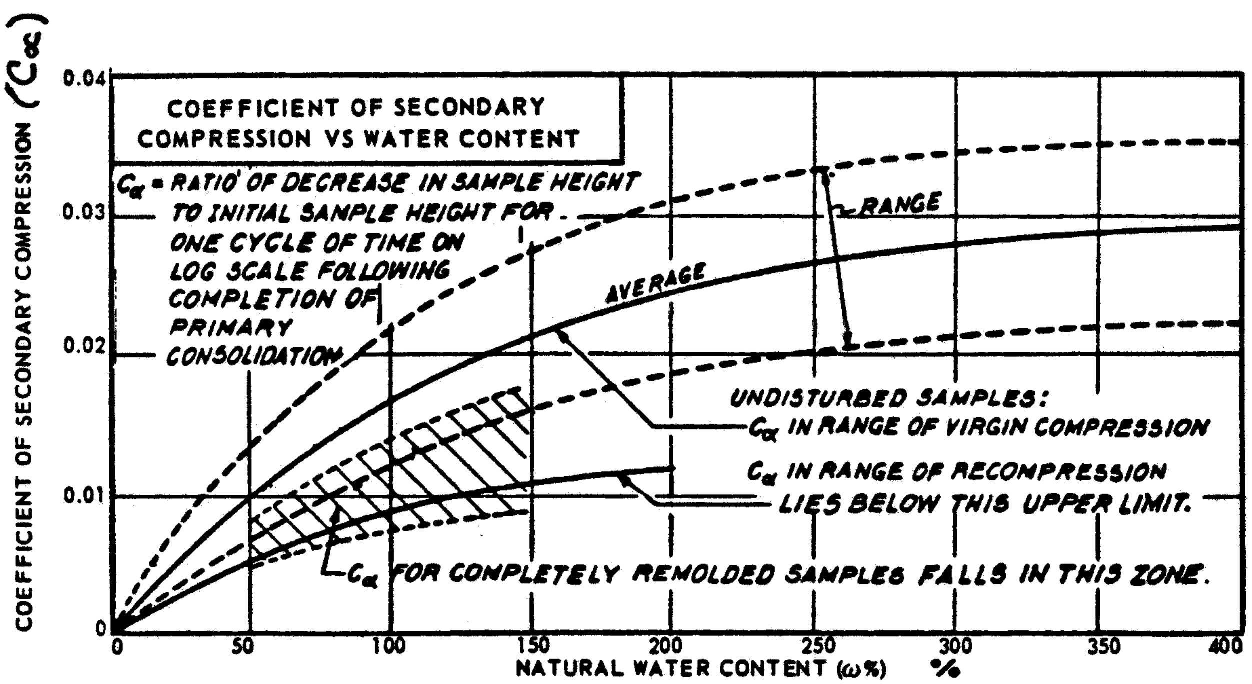


Figure 6-9: Correlation of C_{α} with Natural Water Contents

6.7 TIME RATE OF SETTLEMENT

In practice, settlement amount can be estimated with reasonable accuracy if the settlement estimate is based on properly conducted consolidation tests of quality undisturbed samples.

The time for the primary settlement to occur is more difficult to estimate. Commonly used formulas are based on consolidation of an assumed homogeneous soil deposit; a condition which seldom occurs in nature. Fortunately, however, the computed time estimate will usually be conservative, i.e., settlement in the field usually occurs more rapidly than the time estimated by theory. The reason is that most silt-clay deposits contain more permeable sand or silt lenses which provide lateral drainage and speed the settlement time. Careful attention should be paid during the field investigation to determine if such layers exist in the deposit. All extruded Shelby tube samples should be checked for presence of sand-gravel-silt lenses. Continuous Shelby tube samples taken in at least one boring can be very useful in determining if lenses are present.

The time rate of settlement can be estimated utilizing the coefficient of consolidation (C_v) obtained from consolidation testing. Since the time for 100 percent consolidation to occur is theoretically infinite, the time for 90 percent consolidation is usually considered the total time for primary settlement.

The time rate of settlement is based on a time factor and may be computed from

$$t = \frac{TH_v^{\,2}}{C_v} \qquad (6\text{-}4)$$

Where:
- t = time for settlement to occur (days)
- T = theoretical time factor (dependent on percent consolidation as shown in Table 6-2)
- H_v = maximum length of vertical drainage path in feet (single or double drainage)
- C_v = coefficient of consolidation in feet squared per day (C_v is obtained from the lab consolidation test using the time – compression curve for the test load increment midway between P_o and P_F)

Table 6-2
Time Factor (T)

Percent Primary Settlement	Time Factor (T)
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848

Settlement time can be computed, using the time factors for various percent primary settlements, to develop a predicted time-settlement curve for the field problem.

A typical time-settlement curve for a clay deposit under embankment loading is shown in Figure 6-10.

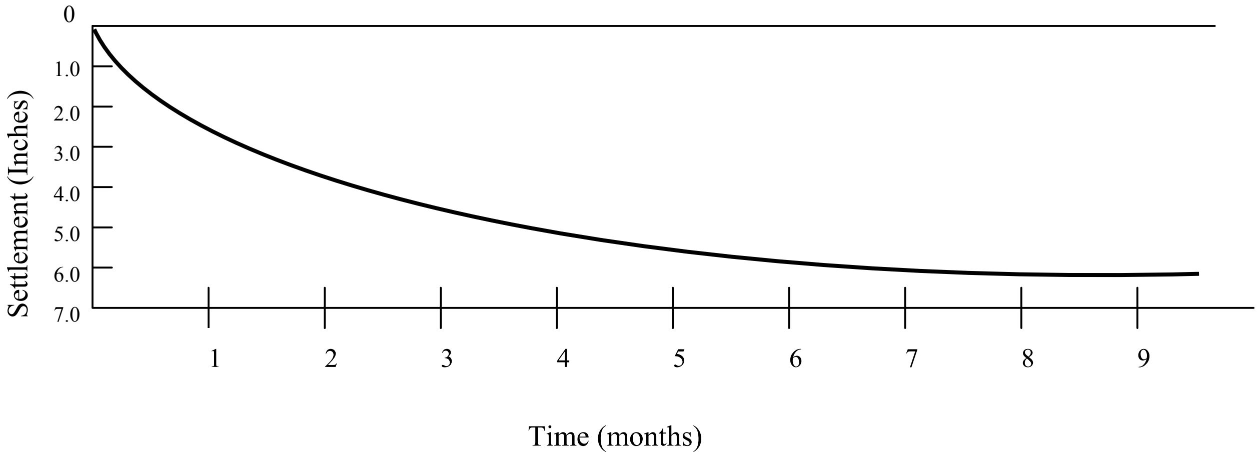
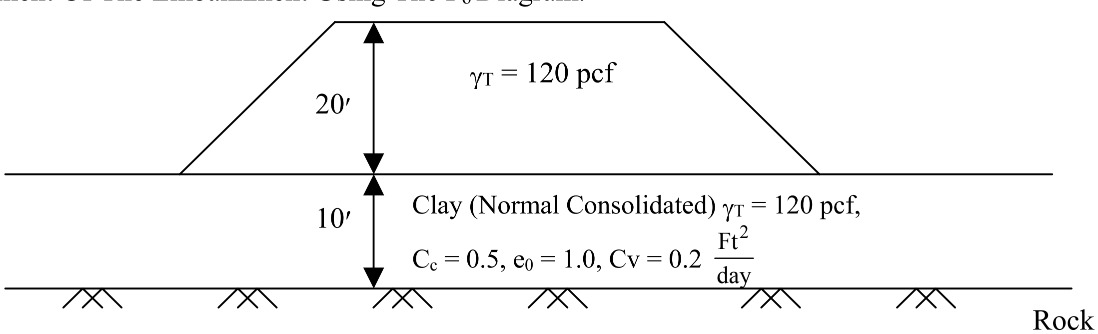


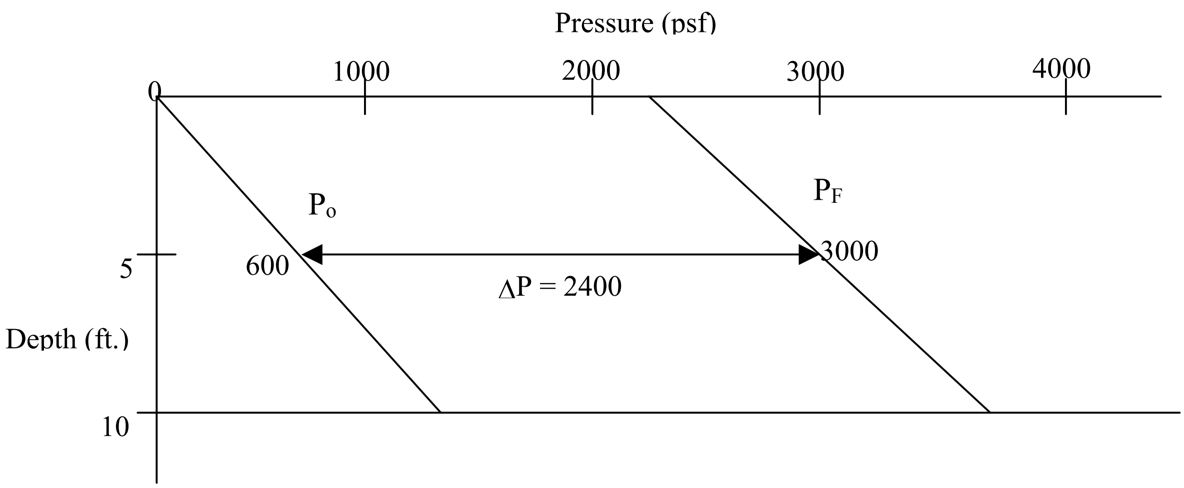
Figure 6-10: Typical Time-settlement Curve for Clay

Important factors to remember are both the time required for consolidation is proportional to the square of the longest distance required for water to drain from the deposit and the rate of settlement decreases as time increases. The maximum length of vertical drainage path, H_v, bears further explanation. This term should not be confused with the H term in the equation for settlement magnitude which is an arbitrarily selected value usually representing a portion of the total compressible layer thickness. The H_v term is the maximum vertical distance that a water molecule must travel to escape from the compressible layer to a more permeable layer. In the case of a 20-foot thick clay layer bounded by a sand layer on top and a non-permeable rock strata on the bottom, the H_v term would equal 20 feet. The water molecule must travel from the bottom of the layer to escape, i.e., single drainage. However, if the clay layer was bounded top and bottom by permeable sand deposits, the H_v distance would be 10 feet. The water molecule in this case, needs only to travel from the center of the layer to either boundary to escape, i.e., double drainage.

Although horizontal drainage considerations are beyond the scope of this manual, the mechanism for determining the maximum horizontal path for escape of a water molecule is similar. The influence of horizontal drainage may be great if the width of the loaded area is small. For instance, during consolidation under a long, narrow embankment, a water molecule can escape by traveling a distance equal to one half the embankment width. However, for very wide embankments the beneficial effect of lateral drainage may be small as the time for lateral escape of a water molecule increases as the square of one-half the embankment width.

Example 6-4: Determine The Magnitude And The Time For 90% Consolidation For The Primary Settlement Of The Embankment Using The P_o Diagram.





Solution:

Find Primary Settlement

$$\Delta H = H \frac{C_c}{1+e_0} Log \frac{P_0 + \Delta P}{P_0} \tag{6-3}$$

$$= 10'\left(\frac{0.5}{1+1.0}\right) Log \frac{600\, psf + 2400\, psf}{600\, psf}$$

$$\Delta \text{H} = 1.75' = 21''$$

Find Time to 90% Consolidation:
Assume Single Vertical Drainage Due to Impervious Rock Layer.

$$t_{90} = \frac{TH^2}{C_v} \tag{6-4}$$

$$t_{90} = \frac{(0.848)(10)^2}{0.2} = 424 \text{ days}$$

6.8 DESIGN SOLUTIONS - SETTLEMENT PROBLEMS

- Reducing Settlement Amount

Several of the methods for solving embankment stability problems can also be used to reduce the amount of settlement. These include:

1. Reduce grade line.
2. Excavate and replace soft soil.

3. Lightweight fill.
4. Stone columns

- Reducing Settlement Time

Often the major design consideration when faced with a settlement problem is the time for the settlement to occur. Low permeability clays and silt-clays can take a long time to consolidate (water squeezed out). The settlement time is generally what will get the chief engineer "excited," since this can affect construction schedules, increase project costs due to inflation, etc. Settlement time is also important to the maintenance forces of a highway agency. The life cycle cost of annual regrading and resurfacing of settling roadways is usually far greater than the cost of design treatments to eliminate settlement during initial construction.

The two most common methods used to accelerate settlement and reduce settlement time are:

1. Surcharge treatment.
2. Vertical drain treatment of subsoil.

6.8.1 Surcharge Treatment

An embankment surcharge is built up a predetermined amount, usually 1 to 10 feet, above final grade elevation and allowed to remain for a predetermined waiting period (typically 3 - 12 months). The actual dimensions of the surcharge and the waiting period will depend on the strength and drainage properties of the foundation soil as well as the initial height of the proposed embankment. The length of waiting period can be estimated using consolidation test data. The actual settlement occurring during embankment construction is then monitored with geotechnical instrumentation. When the settlement with surcharge equals the settlement originally estimated for the embankment the surcharge is removed, as illustrated in Figure 6-11.

The surcharge should not be left on after the desired settlement amount has occurred as additional settlement will occur. Note that the stability of a surcharged embankment must be checked to insure that an adequate safety factor exists to permit placement of the surcharge load.

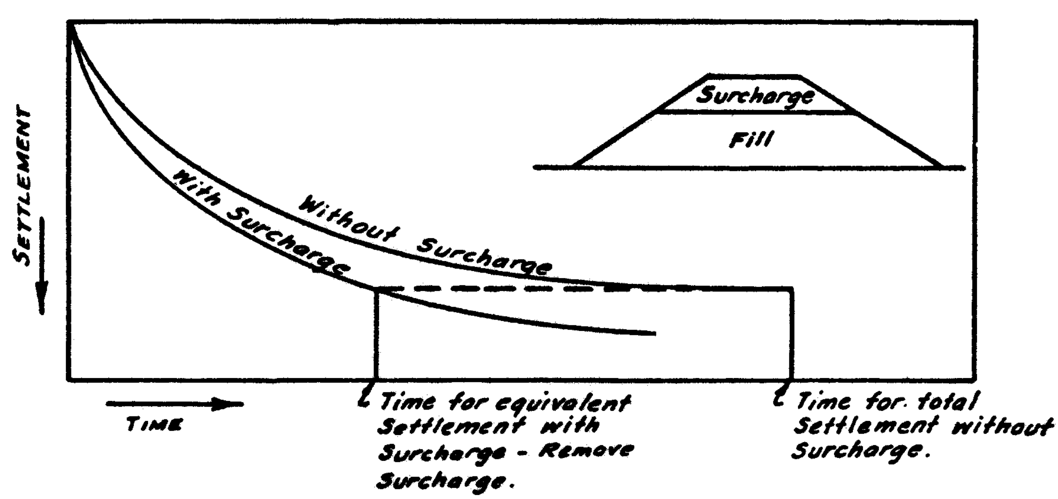


Figure 6-11: Determination of Surcharge Time Required to Achieve Desired Settlement

6.8.2 Vertical Drains

Some highly plastic clays of extremely low permeability can take many years for settlement to be completed. Surcharging alone may not be effective in reducing settlement time sufficiently. In such cases, vertical drains can be used to accelerate the settlement; either with or without the surcharge treatment. Although both sand and wick (prefabricated) types of vertical drains have been used in the past, predominately wicks have been used in recent years due to cost and environmental advantages. For either drain type, a permeable sand blanket, 2-3 feet thick, should be placed on the ground surface to permit movement of water away from the embankment area. The drains are installed prior to placement of the embankment as the pressure to drive the water up the vertical drains is caused by the embankment load. Surcharging should always be considered first, since vertical drains are generally more expensive. The reason vertical drains accelerate the settlement is that the drainage path the water must travel to escape from the impervious soil layer is shortened, as illustrated in Figure 6-12.

Recall that the settlement time is proportional to the square of the length of the drainage path, thus if the drainage path length can be cut in half, the time is reduced by a factor of four. The vertical drains and sand blanket must have high permeability to allow water squeezed out of the subsoil (due to the fill pressure) to travel up the drains and out through the blanket.

Wick drains are small prefabricated drains consisting of a plastic core which is wrapped by a piece of filter fabric. Wick drains are approximately 4 inches wide and about 1/4 inch thick and produced in rolls which can be fed into a mandrel. Wick drains are installed by pushing or vibrating a mandrel into the ground with the wick drain inside. When the bottom of the compressible soil is reached, the mandrel is withdrawn and the trimmed portion of the wick drain left in the ground. To minimize smear of the clay, the cross-sectional area of the mandrel is recommended to be limited to a maximum of about 10 square inches. Preholing of compact surface soil deposits may be required for mandrel installation. Use of wick drains in the United States began about 35 years ago. Wick drain projects will typically be 50 percent less costly than if sand drains were used. This is primarily due to much faster speed of installation and the environmental advantages of wick drains versus sand drains. Wick drains are now used almost exclusively in vertical drain applications.

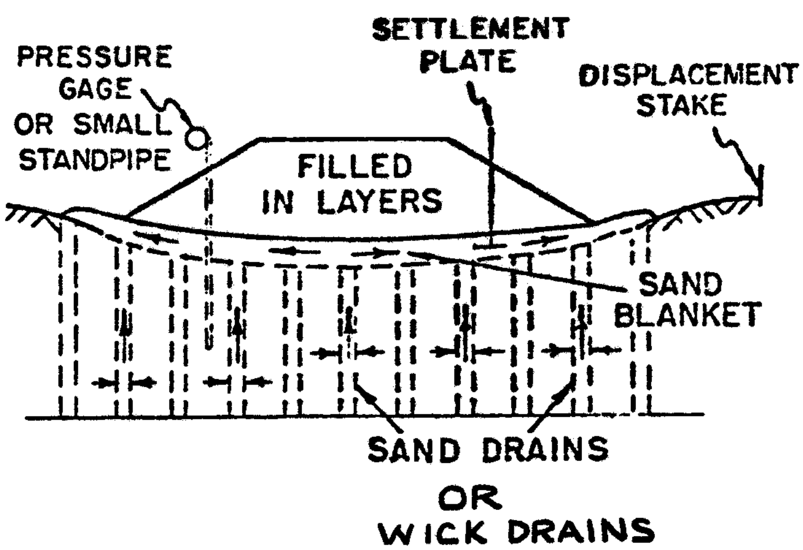


Figure 6-12: Use of Vertical Drains to Accelerate Settlement

6.9 PRACTICAL ASPECTS OF EMBANKMENT SETTLEMENT

Few engineers realize the influence of embankment placement on the subsoils. The total weight of an embankment has an impact on the type of foundation treatment that may be selected. For instance a relatively low height embankment of 10' may be effectively surcharged because the additional surcharge weight could be 30 to 40 percent of the proposed embankment weight. However, when the embankment height exceeds 50' the influence of a 5' or 10' trapezoid of soil on top of this heavy 50' mass is small and probably not cost-effective. Conversely, as the embankment height (and, therefore, weight) increase, the use of a spread footing abutment becomes more attractive. A 30' high, 50' long approach embankment weighs about 15,000 tons compared to the insignificant weight of a total abutment loading which may equal 1,000 tons. Besides weight, the width of an embankment has an effect on total settlement. Wider embankments cause a pressure increase deeper into the subsoil. As might be expected, wide embankments will cause more settlement and will increase the time for consolidation to occur.

Also, the use of geotextiles or geocomposite drains can be an effective method of preventing the bump at the end of the bridge. It is suspected that high dynamic loads are routinely induced in the abutment backfill due to vehicle impact loads. Inadequate filter layers or non-durable drain aggregate can cause either piping of fines or accelerated pavement subsidence due to breakdown of aggregates. In geographic areas where select materials are not available, the use of geosynthetic reinforcement of the abutment backfill and approach area can reduce the bump at the end of the bridge.

Recent developments in microcomputer software now permit simple computer analysis of approach embankment settlement. Programs such as EMBANK permit the user to quickly compute settlements along abutments, piers buried in end slopes or pipes placed diagonally under approach fills.

6.10 APPLE FREEWAY DESIGN EXAMPLE – SETTLEMENT

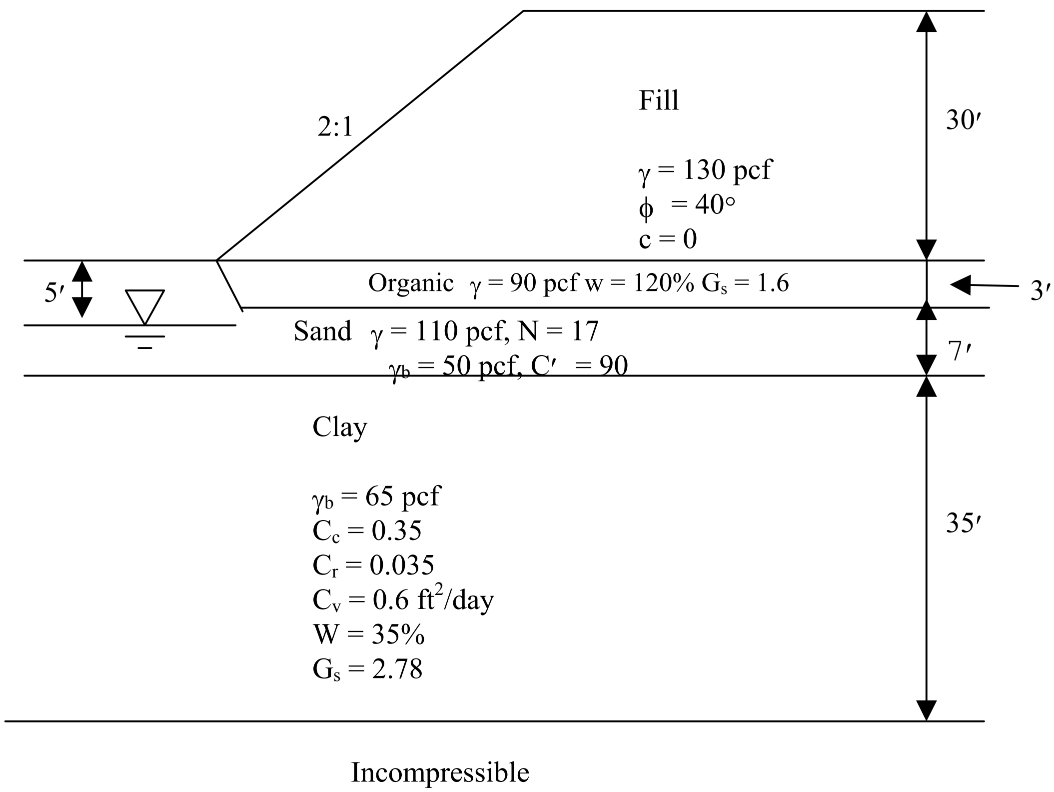
In this chapter the Apple Freeway Example is used to illustrated the computation of settlement and time rate relationship. The options of surcharge and vertical drains are also examined.

Site Exploration	Terrain Reconnaissance Site Inspection Subsurface Borings	
Basic Soil Properties	Visual Description Classification Tests Soil Profile	
Laboratory Testing	Po Diagram Test Request Consolidation Results Strength Results	
Slope Stability	Design Soil Profile Circular Arc Analysis Sliding Block Analysis Lateral Squeeze	
	Embankment Settlement	**Design Soil Profile Settlement Time – Rate Surcharge Vertical Drains**
Spread Footing Design	Design Soil Profile Pier Bearing Capacity Pier Settlement Abutment Settlement Vertical Drains Surcharge	
Pile Design	Design Soil Profile Static Analysis – Pier Pipe Pile H – Pile Static Analysis – abutment Pipe Pile H – Pile Driving Resistance Abutment Lateral Movement	
Construction Monitoring	Wave Equation Hammer Approval Embankment Instrumentation	

Apple Freeway Design Example – Embankment Settlement
Exhibit A

Given: The Subsurface Profile and Soil Properties Shown Below, for the East Approach Embankment of the Apple Freeway Bridge.

Required: Compute the Magnitude and Time-rate of the Anticipated Settlement and Examine the Options of Surcharge and Using Vertical Drains (Including Cost Analysis)



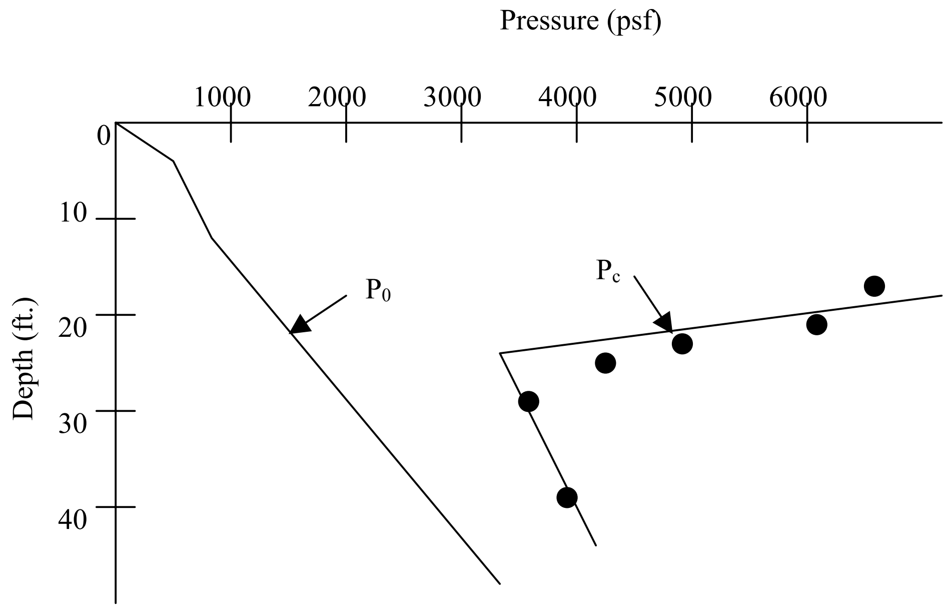
Solution:

Step 1: Obtain Soil Consolidation Characteristics (from lab tests).

CONSOLIDATION TEST RESULTS

Depth	Tube	P_c (psf)	C_c	C_r	C_v (ft^2/day)
11	T3	6500	0.35	0.033	0.6
16	T4	6000	0.32	0.031	0.4
21	T5	4800	0.36	0.040	0.8
26	T6	4200	0.34	0.035	0.6
31	T7	3400	0.34	0.037	0.8
40	T9	3800	0.35	0.032	0.4
e_0 (average) = 0.97					

Step 2: Plot Overburden Pressure and Preconsolidation Pressure Variation with Depth (below)

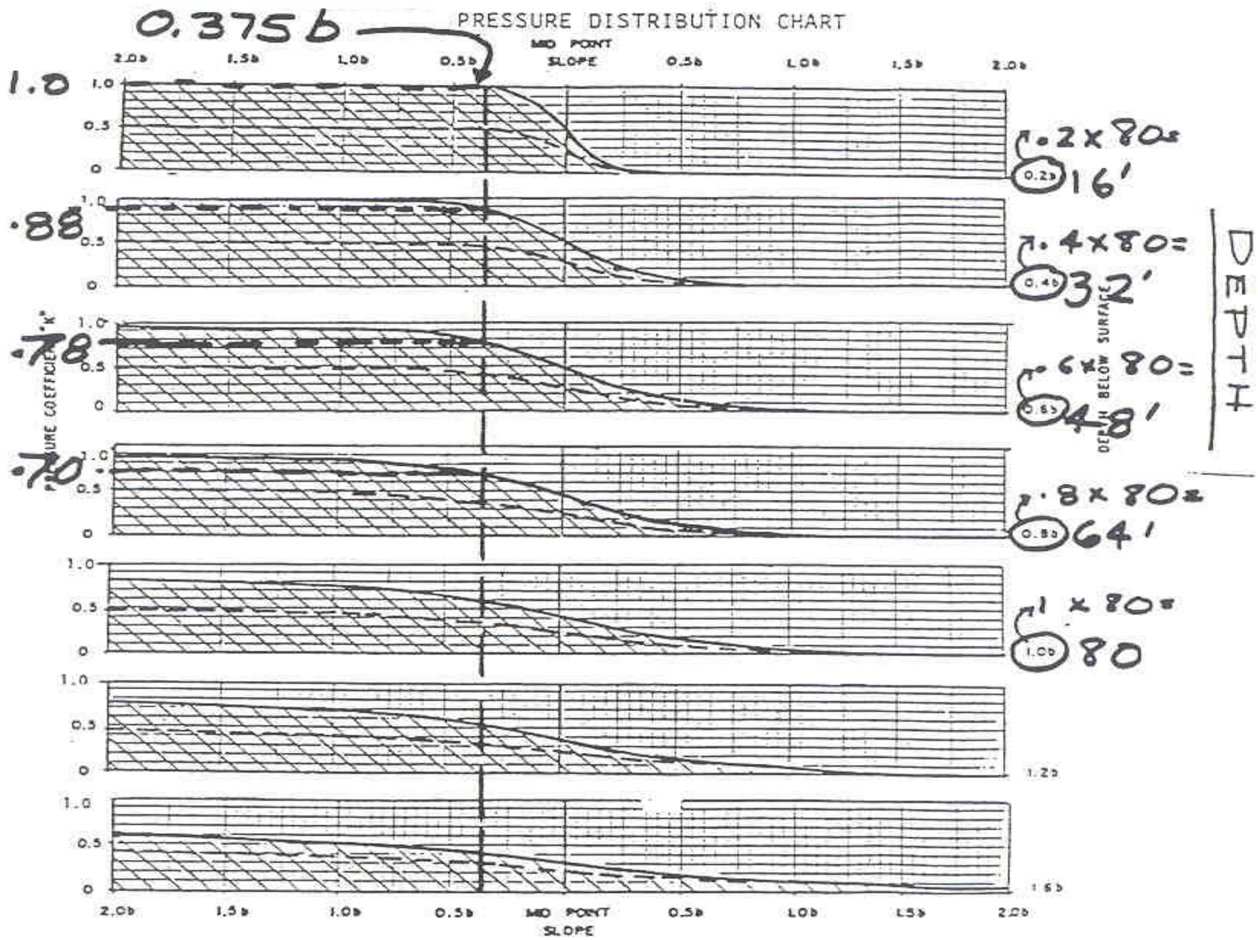


Step 3: Determine Distribution of Final Embankment Pressure (P_F) with Depth:

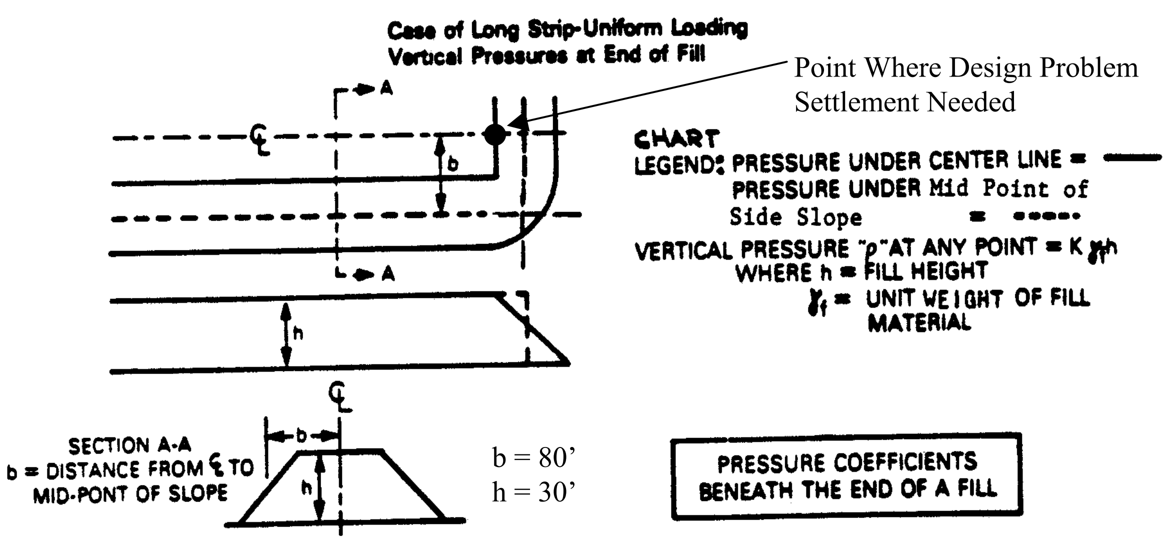
- Obtain embankment geometry (from Plan and Section).
- Embankment top width = 100′
- Side & end slopes 1V on 2H
- Top of end slope 60′ from toe
- Embankment height = 30′
- Embankment load (at center) = $H_{emb} \times \gamma_{emb}$
  $= 30' \times 130$ pcf = 3900 psf

- Abutment center located 30′ from midpoint of end slope $\rightarrow \frac{30}{80}b = 0.375b$
- Go to pressure distribution chart with b = 80′ $\left(\frac{100}{2}+\frac{60}{2}\right)$ and a distance from midpoint of end slope of 0.375b and obtain "K"
- Compute Pressure Change $\Delta P = K \times$ embankment load.

Depth	**″K″**	**ΔP = ″K″ × 3900 Distributed pressure (psf)**
16′	1.00	3900
32′	0.88	3432
48′	0.78	3042
64′	0.70	2730



0.375 b
PRESSURE DISTRIBUTION CHART
MID POINT SLOPE
2.0b 1.5b 1.0b 0.5b 0.5b 1.0b 1.5b 2.0b
1.0
.88
.78
.70
.2 x 80 = 16'
.4 x 80 = 32'
.6 x 80 = 48'
.8 x 80 = 64'
1 x 80 = 80
0.2b
0.4b
0.6b
0.8b
1.0b
1.2b
1.6b
DEPTH
DEPTH BELOW SURFACE
PRESSURE COEFFICIENT "K"

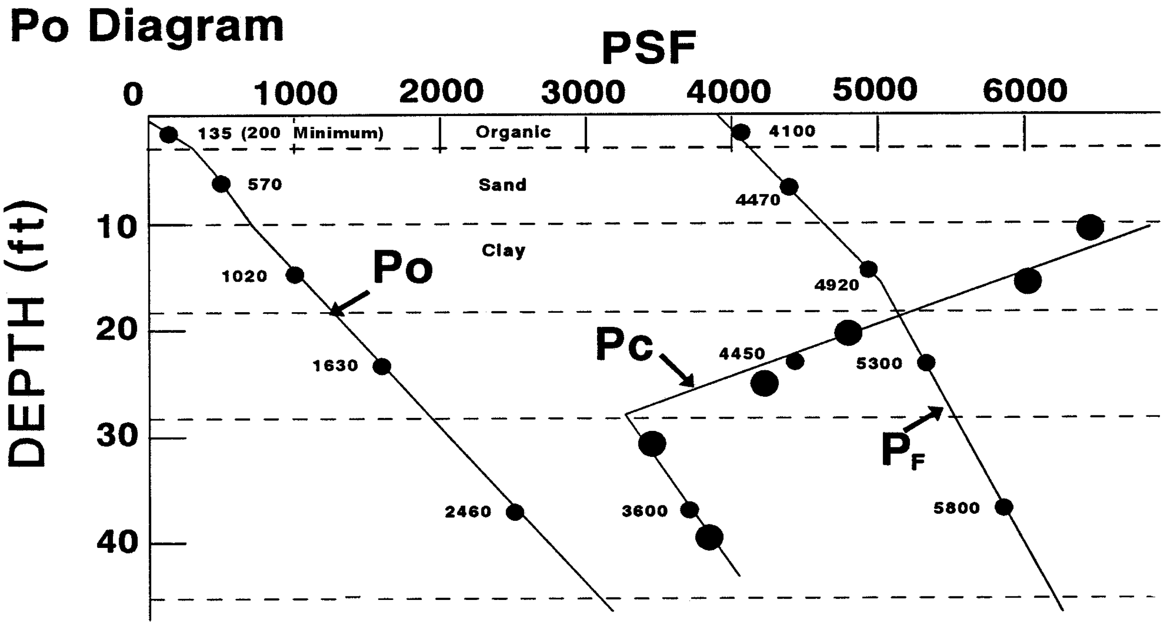


Case of Long Strip-Uniform Loading
Vertical Pressures at End of Fill
Point Where Design Problem Settlement Needed
A
b
h
CHART
LEGEND: PRESSURE UNDER CENTER LINE = ——
PRESSURE UNDER Mid Point of Side Slope = -----
VERTICAL PRESSURE "p" AT ANY POINT = K γ_fh
WHERE h = FILL HEIGHT
γ_f = UNIT WEIGHT OF FILL MATERIAL
SECTION A-A
b = DISTANCE FROM ℄ TO MID-PONT OF SLOPE
b = 80'
h = 30'
PRESSURE COEFFICIENTS BENEATH THE END OF A FILL

Step 4: Plot P_0, P_c and P_F with depth

$P_F = P_0 + \Delta P$

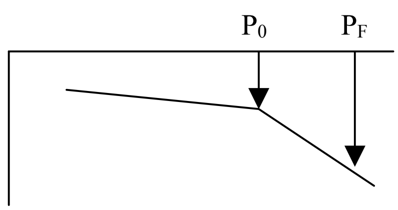
Plot P_F on P_0 diagram



In settlement analysis, use pressures measured at center of layer or partial layer. Thick layers should be subdivided (ie. if layer is 20′ thick. compute settlement in 10′ increments) unless the slope of P_o, P_c, or P_F are slowly converging straight lines. Dashed lines in above diagram show selected increments for analysis.

Step 5: Compute settlement in each layer (or partial layer).

- Layer 1 – Organic (0′ to 3′)



$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_0}$$

H = 3′ - 0′ = 3′

$$C_c = \frac{w}{100} = \frac{120}{100} = 1.2$$

$$e_0 = \frac{w \times G_s}{\%Sat.} = \frac{120 \times 1.6}{100} = 1.9$$

$$\Delta H = 3\left(\frac{1.2}{1+1.9}\right)\text{Log}\frac{4100}{200*} \qquad \text{* Remember } (P_0 \geq 200\text{psf})$$

$$\Delta H = 1.63' = 19.54''$$

- Layer 2 – Sand (3′ to 10′)

$$\Delta H = H\frac{1}{C'}\text{Log}\frac{P_F}{P_0}$$

$H = 10' - 3' = 7'$

To find C' use $N = 17$ (BAF – 3)

$$\frac{N'}{N} = 2 \text{ @ } P_0 = 500\,\text{psf}\,(\text{Figure } 6-5)$$

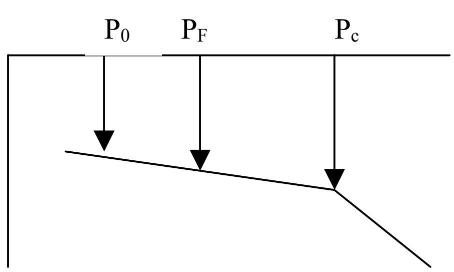
$N' = 34$

$C' = 90$ (Figure 6-6 between silty sand & fine to coarse sand)

$$\Delta H = (7)\left(\frac{1}{90}\right)\text{Log}\frac{4470}{570}$$

$$\Delta H = 0.069' = 0.83''$$

- Layer 3 – Clay (10′ to 18′)



$$\Delta H = H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_0}$$

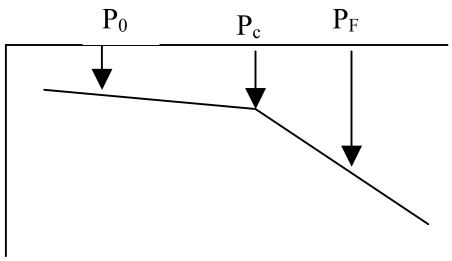
$H = 18' - 10' = 8'$

From Consol. Test data:
C_r (avg.) = 0.035
e_0 (avg.) = 0.97

$$\Delta H = (8)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{4920}{1020}$$

$$\Delta H = 0.097' = 1.17''$$

- Layer 3 – Clay (18′ to 28′)



28′ chosen as P_c slope changes

Compute ΔH separately for $P_0 > P_c$ and $P_c > P_F$

$$\Delta H = H\frac{C_r}{1+e_0}\text{Log}\frac{P_c}{P_0} + H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_c}$$

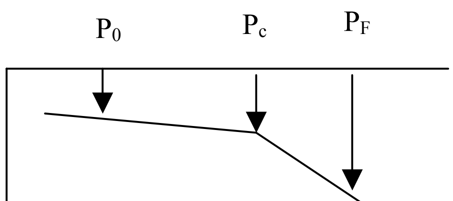
H = 28′ - 18′ = 10′

From Consol. Test data:
  C_r (avg.) = 0.035
  C_c (avg.) = 0.35
  e_0 (avg.) = 0.97

$$\Delta H = (10)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{4450}{1630} + 10\left(\frac{0.35}{1+0.97}\right)\log\frac{5300}{4450}$$

$$\Delta H = 0.077' + 0.135' = 0.93'' + 1.62'' = 2.55''$$

- Layer 3 – Clay (28′ to 45′)



$$\Delta H = H\frac{C_r}{1+e_0}\text{Log}\frac{P_c}{P_0} + H\frac{C_c}{1+e_0}\text{Log}\frac{P_F}{P_c}$$

H = 45′ - 28′ = 17′

From Consol. Test data:
  C_r (avg.) = 0.035
  C_c (avg.) = 0.35

e_0 (avg.) = 0.97

$$\Delta H = (17)\left(\frac{0.035}{1+0.97}\right)\text{Log}\frac{3600}{2460} + (17)\left(\frac{0.35}{1+0.97}\right)\text{Log}\frac{5800}{3600}$$

$$\Delta H = 0.050' + 0.63' = 0.60'' + 7.51'' = 8.11''$$

Total Settlement	
Layer 1 – Organic (0′ to 3′)	19.54″
Layer 2 – Sand (3′ to 10′)	0.83″
Layer 3 – Clay (10′ to 18′)	1.17″
Clay (18′ to 28′)	2.55″
Clay (28′ to 45′)	8.11″
ΔH_{Total}	32.20″

Assume organic layer is excavated and compacted select material placed.

ΔH of 19.54″ in organic layer will be eliminated after excavation of organic layer: ΔH_{Total} = 12.66″.

Step 6: Compute Time for Settlement to Occur

- Layer 1 – Select backfill material no settlement expected.
- Layer 2 – 0.83″ settlement occurs immediately in sand.
- Layer 3 – ΔH = 12.66″ - 0.83″ = 11.8″

Time t computed from: $t = \dfrac{T\,H_V^{\,2}}{C_v}$

H_v = Drainage path
C_v = 0.6 ft^2/day
T = From time factor chart

H_v = ½ thickness of clay layer since permeable layers exist above and below

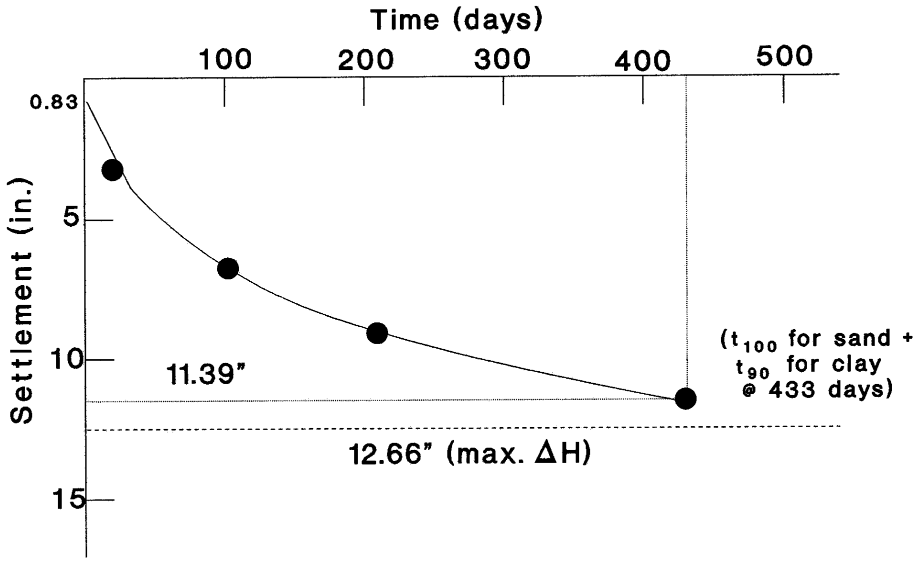
$$H_v = \frac{35'}{2} = 17.5'$$

% Consol. Layer 3	Layer 3 ΔH (in.)	T	$\dfrac{H_v^{\,2}}{C_v}$	t (days)
20	2.4	0.031	510.4	16
50	5.9	0.197	↓	101
70	8.3	0.403	↓	206
90	10.6	0.848	↓	433

The time-settlement plot can now be constructed for all soil layers
ΔH_{Total} = 12.66″

Remember to include 0.83″ sand settlement which occurs immediately as load is applied.

Step 7: Plot Time – Settlement Curve



The designer must insure that 90% consolidation is achieved before construction of the abutment foundation. Choices of treatment are:

1. A 433 day (14 mo.) waiting period
2. Surcharge
3. Vertical drains

Examine Surcharge Option

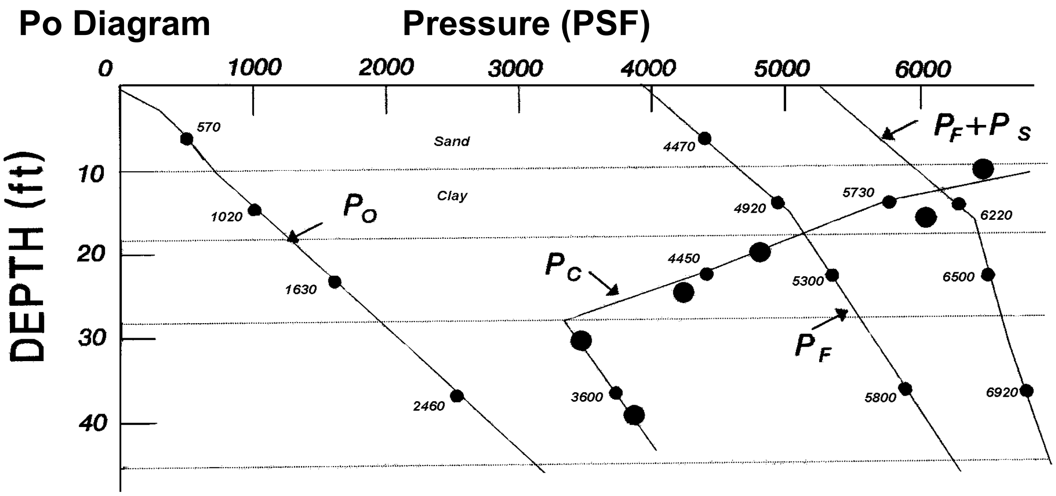
Assume:

- 10′ high compacted surcharged (γ = 130 pcf), ΔP of emb. (P_F) + Surch. (P_s) = 5,200 psf.
- Pressure distribution ″K″ value unchanged, additional consolidation of sand is negligible.
- e_0 remains 0.97 although the actual value is less due to compression under the previous load.

Step 1: Obtain pressure increase with depth (use previous "K" value)

Depth Below OGS	K	K (5200 psf)
0.2b = 16′	1.00	5200
0.4b = 32′	0.88	4580
0.6b = 48′	0.78	4060

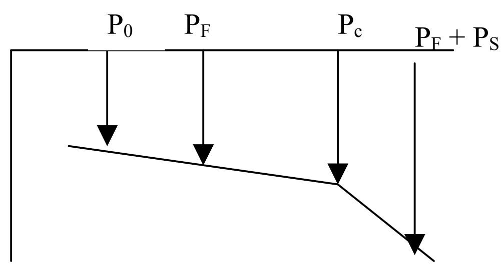
Step 2: Plot Pressure Diagram



Step 3: Compute Settlement in layer 3 (only layer with additional settlement).

Settlement

- Layer 3 – Clay (10′ to 18′)



$$\Delta H = H \frac{C_r}{1+e_0} \mathrm{Log} \frac{P_c}{P_F}$$

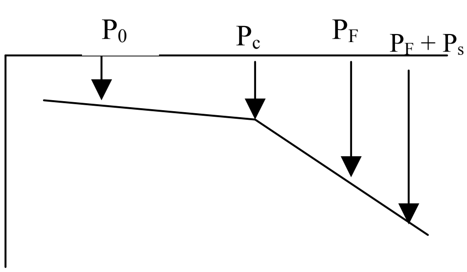
$$\Delta H = (8)\left(\frac{0.035}{1+0.97}\right) \mathrm{Log} \frac{5730}{4920} = 0.01' = 0.12''$$

$$\Delta H = H \frac{C_c}{1+e_0} \mathrm{Log} \frac{P_F + P_s}{P_c}$$

$$\Delta H = (8)\left(\frac{0.35}{1+0.97}\right) \mathrm{Log} \frac{6220}{5730} = 0.051' = 0.61''$$

$$\Delta H = 0.12'' + 0.61'' = 0.73''$$

- Layer 3 – Clay (18′ to 28′)

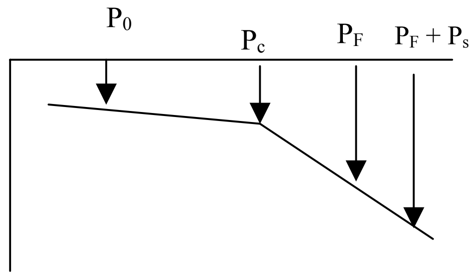


$$\Delta H = H \frac{C_c}{1+e_0} \mathrm{Log} \frac{P_F + P_s}{P_F}$$

$$\Delta H = (10)\left(\frac{0.35}{1+0.97}\right) Log \frac{6500}{5300}$$

$\Delta H = 0.157' = 1.89''$

- Layer 3 – Clay (28′ to 45′)



$$\Delta H = H \frac{C_c}{1+e_0} Log \frac{P_F + P_s}{P_F}$$

$$\Delta H = (17)\left(\frac{0.35}{1+0.97}\right) Log \frac{6920}{5800}$$

$\Delta H = 0.394' = 4.72''$

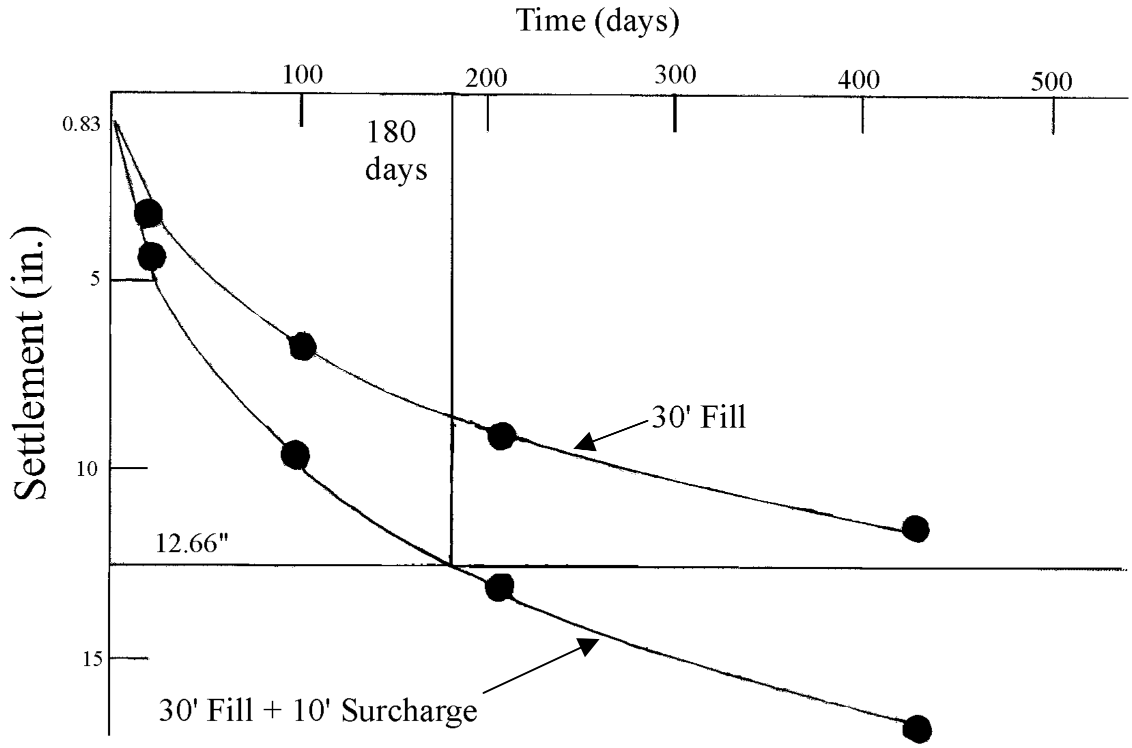
Layer	Embank.	Surch.	Combined
10′ to 18′	1.17″	0.73″	1.90″
18′ to 28′	2.55″	1.89″	4.44″
28′ to 45′	8.11″	4.72″	12.83″
		Total ΔH (clay Layer) =	19.17″

Step 4: Obtain Time-Settlement Relationship: $\mathbf{t = \frac{T\,H_v^{\,2}}{C_v}}$.

%U	ΔH Clay (inches)	T	$\frac{H_v^{\,2}}{C_v}$	T (days)
20	3.8″	0.031	510.4	16
50	9.6″	0.197		101
70	13.4″	0.403		206
90	17.3″	0.848		433

Step 5: Plot time-settlement curve.

Time – Settlement Plot (includes ΔH Sand = 0.83″)



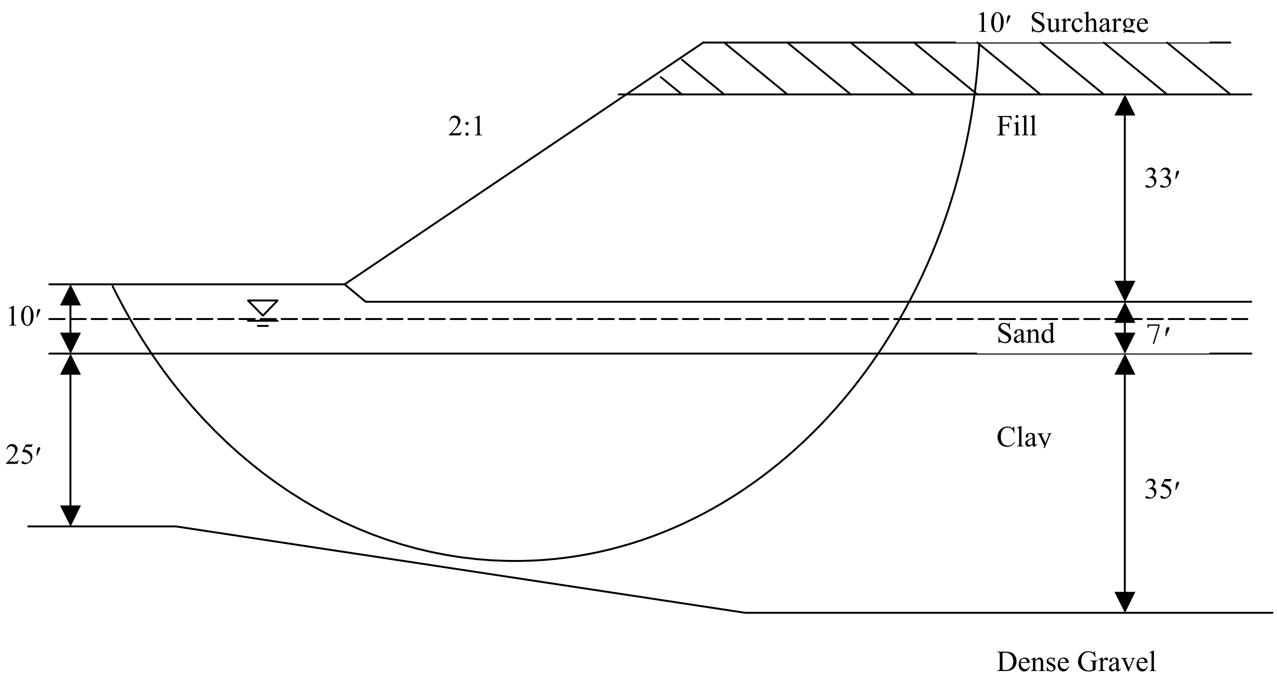
Step 6: **Determine time of waiting period with surcharge to obtain equivalent settlement to that of proposed embankment.**

Enter time – settlement plot for 30′ fill with 10′ surcharge with 12.66″ (settlement expected for 30′ fill). Extend line across to 30′ fill + 10′ surcharge curve and read waiting period time in days from time axis, ie. 180 days or 6 months.

Step 7: **Recommended instrumentation for monitoring settlement:**

Instrument	**Station**	**Depth Below Ground**
Settlement plate	90 + 00	At ground surface
Settlement plate	93 + 50	At ground surface
Settlement plate	96 + 50	At ground surface
Piezometers	93 + 50	20′, 28′, 36′
Piezometers	96 + 50	20′, 28′, 36′

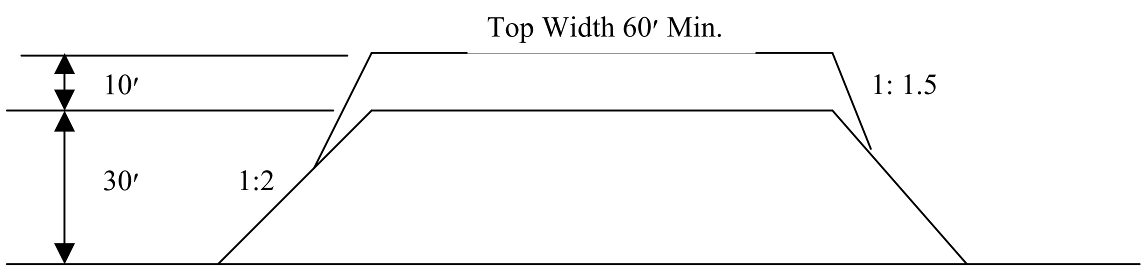
Step 8: **Recheck stability of 30′ fill with 10′ surcharge**



Safety Factor (w/surcharge) = 1.33 (1.63 w/o surcharge)

As safety factor higher than 1.30 (which is minimum recommended for bridge approach stability is O.K)

Step 9: Prepare cost estimate for surcharge



500 linear feet behind top of end slope to be surcharged at each approach.

Total 1000′

Surcharge quantity (avg. width = 80′ including side slopes)

$$80 \times 10 \times \frac{1000}{27} = 29{,}628 \text{ C.Y.}$$

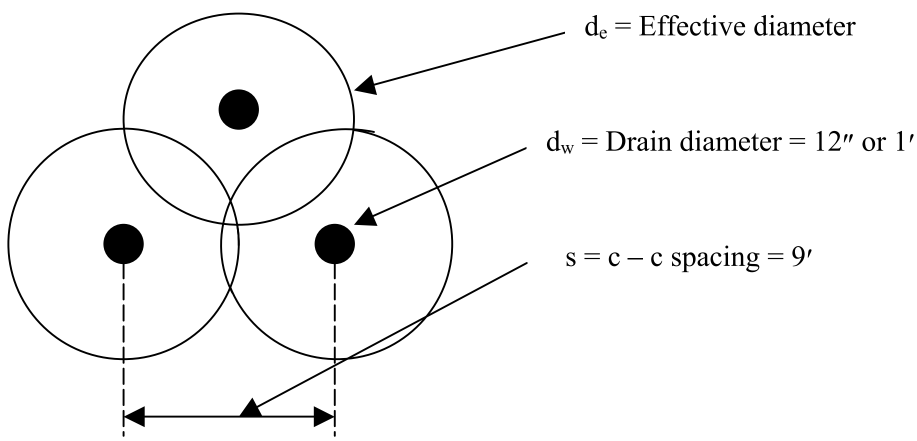
Cost to place and remove surcharge assumed at \$4.00 / C.Y.

Total cost = 29,628 C.Y. $\times$ \$4.00 / C.Y. = \$120,000.00

Examine option of Sand Drains – No Surcharge

Step 1: Choose reasonable spacing of sand drains ie. use 12″ diameter on 9′ center to center triangular spacing.

$C_v = 0.6\ ft^2/day$, assume $C_H = 0.6\ ft^2/day$ also.



Step 2: Compute time-settlement relationship

For triangular spacing: $d_e = 1.05 \times s$

$= 1.05 \times 9' = 9.5'$

$\eta = d_e/d_w = 9.5' / 1' = 9.5'$ (Say 10′)

Use time factor curves for radial drainage such in FHWA-RD-86-168; Figure 4.

Sand Drains – No surcharge

U_R % Consolidation	T_R = (Radial Time Factor)
20	0.045
50	0.140
70	0.230
90	0.450

- Check t_{90} for radial drainage to see if assumed 9′ spacing is effective

$t_{90} = T_R\, d_e^2/C_H$
$= (0.45)(9.5)^2/(0.6) = 68$ days
OK (t_{90} w/o drains = 433 days)

- Check time – settlement for combined vertical and radial drainage.

U_C = % consolidation for combined drainage = 100% - [(100% - U_R)(100% -U_V)]

Assume time (in days) to compute U_C

- Check for t = 30 days

$T_R = t\, C_H/d_e^2 = (30)(0.6)/(9.5)^2 = 0.20$

U_R = 64% (from FHWA-RD-86-168; Figure 4, Radial Flow)

$H_V = ½\, H = 17.5'$

$$T_V = t\frac{C_V}{{H_V}^2}$$

$$T_V = (30)\frac{0.6}{17.5^2} = 0.06$$

U_V = 28% (Estimate from FHWA-RD-86-168; Vertical Flow)

$U_C = 1.00[(1.00 – 0.64)(1.00 – 0.28)]$
U_C = 0.74 or 74%

Settlement of layer 3 @ 74%
$\Delta H = (0.74)(11.8'') = 8.7''$ @ 30 days

- Check for t = 68 days

$T_R = t\, C_H/d_e^2 = (68)(0.6)/(9.5)^2 = 0.45.$

U_R = 90%

$$T_V = t\frac{C_V}{{H_V}^2} = (68)\frac{0.6}{17.5^2}$$

U_V = 48%

$U_C = 1.00[(1.00 – 0.90)(1.00 – 0.48)] = 0.95$

- Settlement of Layer 3 @ 95%
$\Delta H = (0.95)(11.8'') = 11.2''$ @ 68 days

Examine Option of Wick Drains – No Surcharge

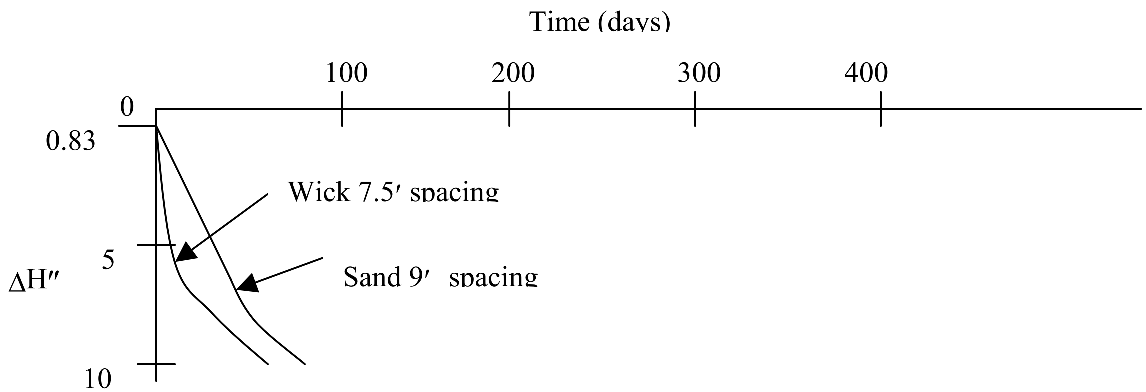
Step 1: Assume equivalent sand drain diameter and perform analysis as for sand drains.

Recent designs for wick drains have used equivalent diameters of 10 – 15 cm

$d_{equivalent}$ = 15 cm or 0.5′ = d_w
C_v = C_{radial}

Try 7.5′ center to center spacing triangular d_e = 1.05(7.5′) = 7.9′

t_{days}	U_V%	U_R%	U_C%	**Layer 3 ΔH″**
10	16	34	44	5.2
20	22	57	66	7.8
30	27	71	79	9.3
40	32	81	87	10.3
50	33	85	90	10.6
60	39	92	95	11.2



Step 2: Prepare cost estimate for vertical drains

Assume:

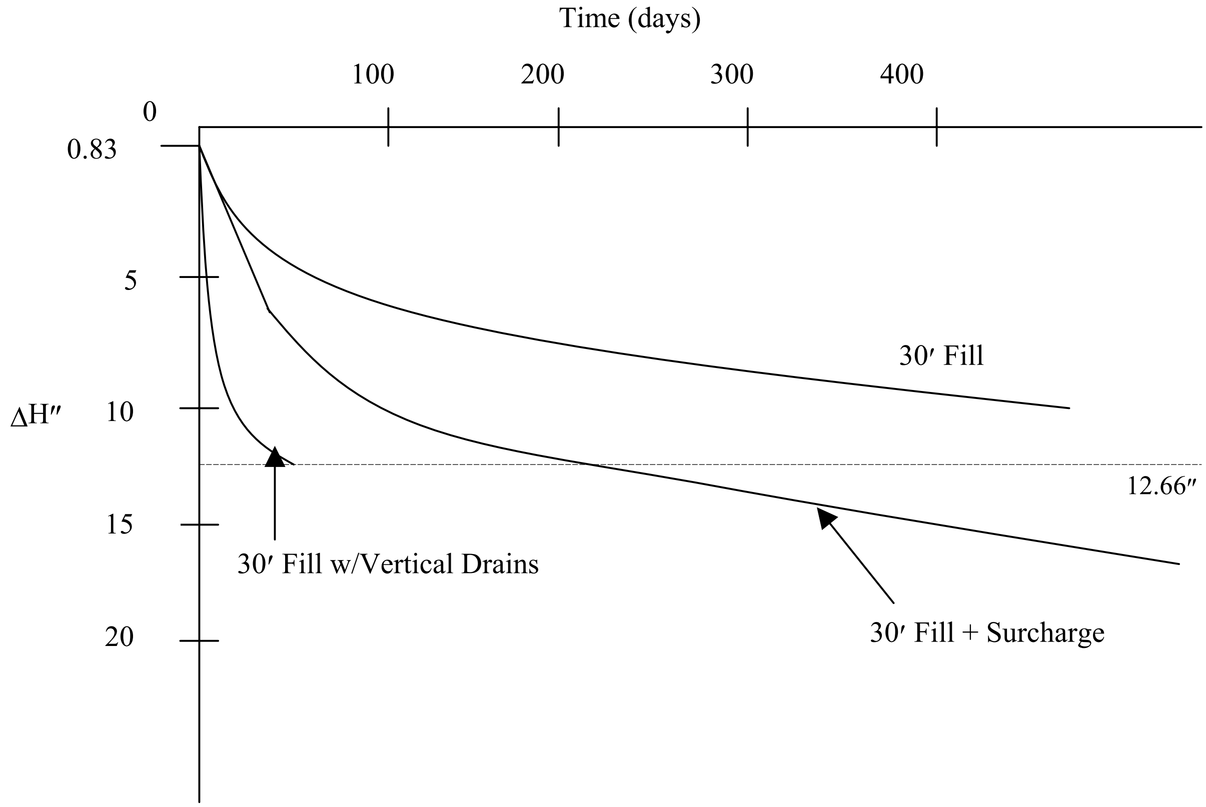
1. 500 L.F. of drains @ both approaches: Total 1000 L.F.
2. Width of drain treatment midslope to midslope: Total 160 L.F.
3. Length of drains: each 45 feet
4. Unit cost: per each – Sand \$3.50/ft
   Wick \$1.00/ft

- Sand Drains 9′ C – C

  Treated area/drain = 0.866 S^2 = 0.866$(9)^2$ = 70 S.F.
  No. of drains = (160)(1000)S.F./70 S.F. = 2286
  Linear feet of drain = (2286)45 = 102,870 L.F.
  Cost = (102,870)(\$3.50) + \$25,000(Mobilization) = \$385,000

  Estimated production at 1800 L.F. per day for 1 rig = 57 days construction time.

- Wick Drains 7.5′ C – C
  Treated area/drain = $(0.866)(7.5)^2$ = 49 S.F.
  No. of drains = 160,000/49 = 3265
  Linear footage = (3265)(45) = 146,925 L.F.
  Cost = (146,925)($1.00) + $25,000 (Mobilization) = $172,000



Time – settlement relationship for (a) 30′ fill, (b) 30′ fill with vertical drains and, (c) 30′ fill and surcharge

Treatment	**t Months**	**Extra Cost**
Fill Only	14	-
Fill w/10′ Surcharge	6	$120,000
Fill w/Wick Drains	2	$172,000
Fill w/Sand Drains	2	$385,000

Estimate amount of horizontal abutment movement due to lateral squeeze of clay.

Rule of Thumb:

Horizontal Movement = 0.25 ΔH of embankment (in clay)
= 0.25 × 11.8″

Horizontal Movement = 3″

Recommend no spread footing construction or pile driving unit settlement nearly complete (t_{90}).

Summary of the Embankment Settlement Phase for Apple Freeway Design Problem

- Design Soil Profile

  Soil layer consolidation properties selected.

- Settlement

  32" of settlement predicted
  19.5" in organic
  0.8" in sand
  11.8" in clay.

- Time-Rate

  433 days for $T_{90.}$

- Surcharge

  10′ surcharge improves t_{90} to 180 days
  cost $120,000. F.S. w/ surcharge = 1.33 O.K.

- Vertical Drains

  60 days for t_{90}
  cost between $172,000 and $385,000